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RECLAIMING HYPERION EFFLUENT

By Finley B. Laverty,[1] F. ASCE, Ralph Stone,[2] F. ASCE, and Lawrence A. Meyerson[3]

SYNOPSIS

The results of approximately three years of tests performed by the Los Angeles County Flood Control District utilizing prototype facilities to reclaim city of Los Angeles Hyperion Treatment Plant effluent for injection into a recharge well are described and summarized. Various secondary and tertiary polishing treatment processes were investigated as methods of reclaiming Hyperion Plant high-rate and standard-rate activated sludge effluent. Results achieved by tertiary treatment involving continuous and intermittent land spreading of effluent, rapid sand filtration, and other processes, including chlorination, are reviewed. These test results indicate the feasibility of reclaiming waste water at reasonable cost, by removal of sufficient suspended matter, dissolved organic substances, and bacteria to enable a test recharge well to be operated satisfactorily in a manner similar to that achieved with a filtered potable water supply. Detailed hydraulic, physical, mineral, organic, bacterial, and microscopic tests were performed. Special studies on nitrogen and detergent control and public health hazards of the injected water were also completed, and the hydraulic characteristics of the well were determined. Fur-

Note.—Discussion open until April 1, 1962. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SA 6, November, 1961.

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2 Cons. Engr., Ralph Stone and Co., Inc., Engrs., Beverly Hills, Calif.
3 Senior San. Engr., Calif. Regional Water Pollution Control Bd. No. 4, Los Angeles, Calif.

ther study of tertiary processes is needed, particularly of rapid sand filtration and other high-rate systems.

INTRODUCTION

Prior to the Hyperion reclamation and recharge test, the Los Angeles County Flood Control District (LACFCD) conducted other local reclamation investigations.[4,5] In each case an essentially uniform effluent, derived from sewage primarily domestic in nature, was utilized.

The first of these was undertaken in 1948, at Whittier, Calif., in Los Angeles County, using trickling filter effluent from the Whittier sewage treatment plant. The plant effluent was high in quality, approximating 10 ppm 5-day BOD and 30 ppm suspended solids. It was spread continuously to a depth of 0.5 ft - 1.0 ft in a small test basin having a bed of fine, sandy loam. After 7 days the percolation rate (here used synonymously with "seepage" or "infiltration" rate) had fallen from 0.75 cfs per wetted acre (cfs per wa) to 0.30 cfs per wa. Examination of the percolate recovered from sampling pans located 4 ft, 5 ft, 6 ft, and 7 ft below the basin surface demonstrated that the degree of polishing increased with depth. At the 7-ft pan the suspended solids averaged only 1 + ppm, and the BOD and B.coli had been completely removed.

The District's second study was performed the following year (1949), at the city of Azusa, Calif. Trickling filter effluent from the Azusa plant, averaging 35 ppm BOD and 61 ppm suspended solids, was spread in a small plot located in coarse, non-uniform, highly permeable sand and gravel. A percolation rate averaging 0.6 cfs per wa under a head of 1.0 ft was maintained for two months. Samples of percolate, from a pan located 7 ft below the basin surface, averaged 11 ppm suspended solids and 2.5 ppm BOD. Removal of coliform bacteria was nearly complete.

Both of these tests indicated that satisfactory percolation rates could be maintained by spreading continuously for periods of 7 days to 30 days, depending on soil permeability, followed by bed rehabilitation by means of drying and cultivation.

Disposal at Azusa has been practiced continuously in the preceding manner since 1949. Water pumped from nearby wells has never shown signs of contamination or pollution.

Test Aims.—The Hyperion effluent reclamation investigation originated as a consequence of severe overpumping of the West Coast Ground Water Basin in Los Angeles County, the western boundary of which is the Pacific Ocean. Reversal of the normal ground water gradient has resulted in sea water intrusion into the coastal portions of the basin.

The LACFCD is charged with the responsibility of water conservation in this area. It has initiated and developed a program to create a fresh-water recharge mound by injecting imported and treated Colorado River water purchased from the Metropolitan Water District of Southern California (MWD)

4 "Land Disposal of Sewage and Industrial Wastes," by R. Stone, Journal of Sewage and Industrial Wastes, Assoc., Vol. 25, No. 4, April, 1953.

5 "Sewage Reclamation by Spreading Basin Infiltration," by R. Stone and W. Garber, Transactions, ASCE, Vol. 117, 1952, p. 1189.

into strategically located wells parallel to and some 2,000 ft inland from the coast.[6] This mound serves both as a barrier against further intrusion by reversing the direction of flow and as a source of ground water replenishment.

Softened and filtered MWD water has been injected continuously at relatively low pressures (about 50 ft - 150 ft head) since 1953. The cost of this water to the District is $22 per acre-ft. The increasing demand for water in southern California indicates that the reclamation of waste water, if it can be performed at a reasonable cost, will be a valuable contribution to the basic economy of the area. The water currently used for recharge purposes would then be available for such higher applications at domestic and industrial water supply at locations where its characteristics create a special value. The reclamation of Hyperion waste water thus presents the dual possibilities of reducing the initial cost of the recharge water for the salt water barrier program and conserving MWD potable waters for higher beneficial uses.

The Hyperion reclamation test program has also enabled the collection of pertinent information concerning the general operating problems associated with reclaiming waste water, the quality of reclaimed water achieved with various methods of treatment, the extent of odor and esthetic nuisances, if any, that might occur, the control of ground water pollution and contamination hazards, and preliminary cost information on construction and operation of water reclamation facilities.

It was originally planned to utilize available land areas located adjacent to the Hyperion Treatment Plant, in the vicinity of the city of Los Angeles International Airport, as a water reclamation site. It was postulated that the Hyperion effluent could be spread on surface spreading grounds and the reclaimed percolated water would be stored within a controlled perched aquifer. Conventional water well pumps and a 3-mile force main could then be used to supply this water to the barrier distribution system at a head of about 300 ft. For these purposes, it was deemed necessary to develop and demonstrate, by actual field tests, a suitable economical method for producing a reclaimed water of satisfactory mineral, bacterial, and organic quality that could be injected into recharge wells.

Unfortunately, within two years after these tests were undertaken, the Los Angeles City Airport Commission found it necessary to purchase much of the proposed land area that was to be used as the surface spreading grounds site. Investigation indicates there will be little likelihood of locating other suitable land area in the vicinity of the Hyperion Plant for installation of the several hundred acres of spreading grounds needed.

Following this development, the District's test program was altered to face the new objective of developing a low-cost, high-rate treatment method for reclaiming Hyperion effluent that would require minimum land area and yet provide suitable water quality for recharge well operation. Pumping and transmission facilities would be needed to carry this water to the main line of the salt water barrier distribution system.

Consequently, the initial studies reported herein were directed toward reclamation by land spreading, and the later tests toward high-rate tertiary treatment to polish the plant effluent for direct injection into recharge wells.

---

6 "Development of Fresh-Water Barrier in Southern California for Prevention of Sea Water Intrusion," by Finley B. Laverty, and H A. van der Goot, Journal, AWWA, Vol 47, No. 9, September, 1955, p. 886.

Scope of Investigations.—This paper attempts to present a summary of the general results of the Hyperion reclamation test program based on numerous field investigations and thousands of laboratory tests (Approximately 85,000) completed during the continuous study period extending from about January 1, 1955, through January 31, 1958.

The study attempted to determine the following:

1. The most favorable conditions for reclaiming Hyperion effluent by spreading;
2. The degree of tertiary treatment required for successful injection;
3. Practical means of accomplishing tertiary treatment;
4. The effect of recharge of reclaimed sewage on ground water quality; and
5. A cost study of the factors involved.

Initially, the Hyperion tests were based on the experience derived from the previously described Azusa and Whittier tests, but it soon became apparent

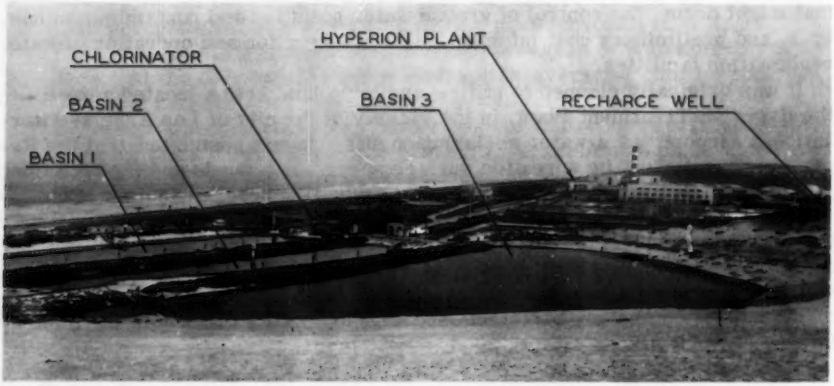


FIG. 1.—GENERAL VIEW OF HYPERION TEST SITE

that it would be necessary to develop different spreading methods in order to achieve both the percolation rate of 1 cfs per wa and the reclaimed water quality necessary for well injection. Unlike the Whittier and Azusa sewage, the Hyperion effluent is subject to considerable variations in quality due to presence of industrial wastes, ground water infiltration, and a lengthy collecting sewer system that causes marked fluctuations in temperature and septicity during warm-weather months.

General Description of Test Facilities.—The test project was located, as shown in Fig. 1, on a low hill overlooking the Hyperion Treatment Plant, Effluent from the plant's high-rate activated sludge treatment process was pumped to the test site. During the latter stages of the investigation, conventional standard-rate activated sludge treatment was approximated in order to study its effect on tertiary treatment. This was done in anticipation of a pending change

in the Hyperion Plant treatment process. Following installation of lengthy ocean outfalls for a sludge and effluent disposal, the plant will revert to primary treatment. Sufficient flows of sewage, however, will undergo standard-rate activated sludge treatment in the existing facilities to more than supply the reclamation needs of LACFCD for barrier purposes.

Fig. 2 depicts a schematic diagram and flow sheet of the principal facilities used in the project. These included a pumping plant to supply Hyperion effluent to the site, two test sand spreading basins used for prototype water spreading

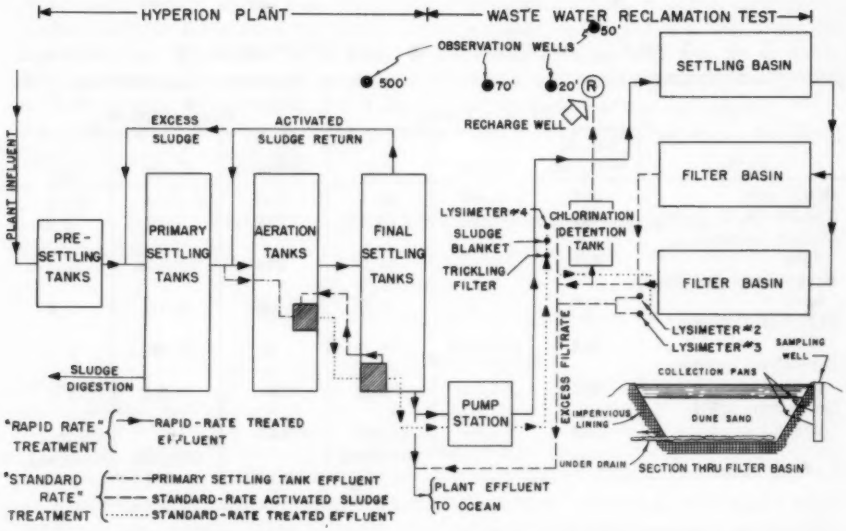


FIG. 2.—HYPERION TEST SCHEMATIC FLOW DIAGRAM

operations and described in this report as Basins Nos. 1 and 2, a storage reservoir described as Basin No. 3, a chlorine contact tank and chlorination equipment, several weir boxes to measure flows, a test recharge well, and a series of observation wells. Additional test facilities included special lysimeters and trickling filter and rapid sand filter pilot plants.

EXAMINATION OF TEST OPERATIONS AND RESULTS

Location of Major Sampling Stations.—During the course of the test numerous chemical, bacteriological, organic, and other laboratory samples were obtained from many points within the test system as shown in Fig. 2. The major sampling stations were as follows: The effluent from the Hyperion Treatment Plant, the effluent from the storage basin (No. 3), the filtrate from the spreading and filter basins (Nos. 1 and 2), the inflow into the recharge test well, and

water pumped from the observation wells (described by their linear distance in feet from the recharge well as observation wells HR-50, HR-70, and HR-500 respectively). Composite effluent samples were obtained where needed. These samples were collected in accordance with standard methods[7] or other accepted engineering practices. The refrigerated samples were promptly transported for analysis either at the test site or at the Hyperion and LACFCD laboratories. Certain detergent analyses were performed at the laboratory of the Los Angeles County Sanitation Districts. Standard analytical techniques, where applicable, were utilized to develop the chemical, bacterial, microscopic, and other test data.

TABLE 1.—HYPERION PLANT EFFLUENT QUALITY

Test	HRAS Effluent			SRAS Effluent		
	No. of Samples	Range	Avg.	No. of Samples	Range	Avg.
BOD, ppm	363	5-90	47.2	250	0.5-68	12.0
Soluble BOD, ppm	55	6-40	18.4	278	0-32	2.7
Suspended Solids, ppm	351	10-158	56.0	254	5-116	31.3
Fixed Susp. Solids, ppm	10	0-35	6.6	69	0-16	3.6
Chlorine Demand, ppm	222	1.4-36.8	16.8	96	1.0-19.3	3.5
Dissolved Oxygen, ppm	302	0.0	0.0	140	0-3.8	1.0
Coliform, MPN per ml	353	240-2,400,000	70,000 (median)	163	46-240,000	6,200 (median)

Basin Influent Quality Summary.

Hyperion Effluent.—The Hyperion high-rate activated sludge plant effluent was utilized in the test program from February 24, 1955, through March 22, 1957. During the final months of the investigation (March 22, 1957 through January 31, 1958), the inflow from Hyperion was altered from the normal high-rate activated sludge (HRAS) to a standard-rate activated sludge (SRAS) effluent by conversion of a plant aeration tank and a final clarifier unit to experimental use. The Hyperion staff established and maintained proper conditions of aeration and sludge return.

A comparison of the quality of effluent from the high-rate and standard-rate activated sludge processes obtained during the test and after establishment of equilibrium in the latter process is shown in Table 1.

Storage Reservoir (Basin No. 3).—After preliminary tests, Basin No. 3 was constructed and Hyperion effluent supplied to it during certain phases of subsequent tests.

7 "Standard Methods for the Examination of Water, Sewage, and Industrial Wastes," 10th Edition, Amer. Pub. Health Assoc., 1955.

This reservoir encompassed about one acre of surface area and provided a limited oxidation pond that permitted additional sedimentation as well as biological oxidation. The basin contained a storage volume varying from 2 acre-ft to 4 acre-ft and was normally operated with an average theoretical detention time of some 2 days to 4 days. The actual detention time, as shown by dye tests, was about half the theoretical due to short-circuiting of the flow. The 5-day BOD, suspended solids, and coliform averages obtained form Basin No. 3 during the Hyperion tests are shown in Table 2. When the basin was used, its effluent was supplied directly ot other test operations.

Basin Spreading Operations.—Two half-acre spreading basins (Nos. 1 and 2) were constructed at the test site. Each overlay a 13-ft depth of well-sorted highly uniform dune sand having an effective size of 0.5 mm and equipped with underdrains. Under the dune sand a more impervious soil was treated with dry cement and bentonite to make it watertight, and side dikes were similarly formed to give a freeboard of 3.5 ft. Sampling pans installed at depths of 3 ft, 5 ft, 7 ft, and 10 ft below the basin surface drained to sampling wells located at the periphery of the basins.

TABLE 2.—OXIDATION POND PERFORMANCE

Test	HRAS Effluent			SRAS Effluent		
	Inflow	Outflow	Reduction, in percentage	Inflow	Outflow	Reduction, in percentage
BOD, ppm	48.4	30.2	37.6%	22.8	17.4	23.7%
Susp. Solids, ppm	58.6	51.2	12.6%	63.0	51.4	18.4%
B. Coli, MPN per ml	240,000	62,000	74.1%	6,200	2,300	62.9%

Following preliminary conditioning of the basin soils with city water, application of the sewage plant effluent was initiated. The chronology, BOD loadings, suspended solids, and percolation rates of the various tests that were performed in Basin 1 are shown in Figs. 3 through 5, and the corresponding data are illustrated for Basin 2 in Figs. 6 through 8.

Within a few days after sewage effluent was introduced, a dark gray slime or "Schmutzdecke" was observed to have coated the basin surface. Even though at this time the sewage was being diluted with an equal volume of fresh water (see Fig. 3) and contained an average of 5.5 ppm of dissolved oxygen, an anaerobic mat formed in which the odor of hydrogen sulfide was detectable and thiobacteria were later identified in large numbers.

The significance of the mat as constituting the control surface determining the rate of flow quickly became evident. By the seventh week of the test, when application of undiluted HRAS effluent to Basin 1 began, the percolation rate had dropped from an initial value of 4.6 cfs per wa to 0.5 cfs per wa. Two months later it was less than 0.2 cfs per wa. The quality of water obtained during this phase of continuous spreading is shown in Table 3. Later, Basin No. 3 was used in an attempt to achieve further stabilization of the Hyperion effluent before spreading. The results are shown in Table 3.

A series of lysimeter tests demonstrated that an increase in liquid depth to as much as 19 ft failed to improve percolation rates significantly. Addition of

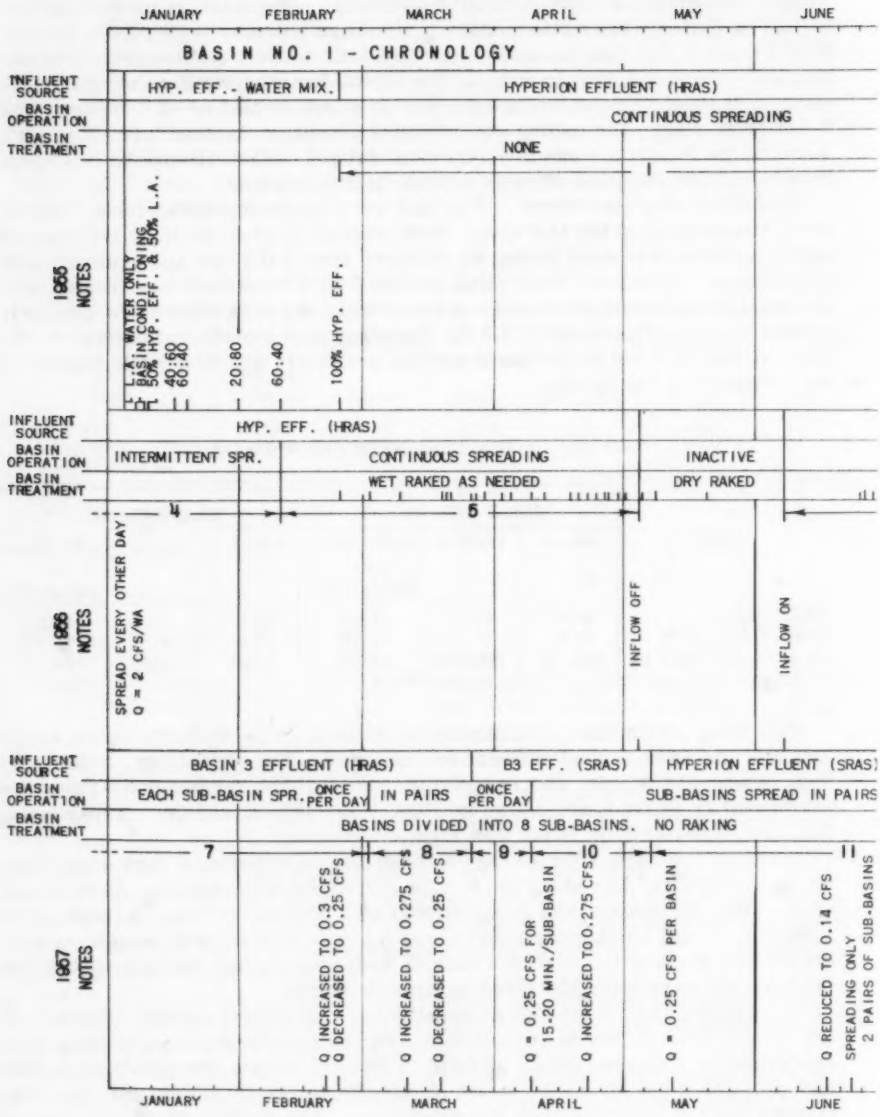
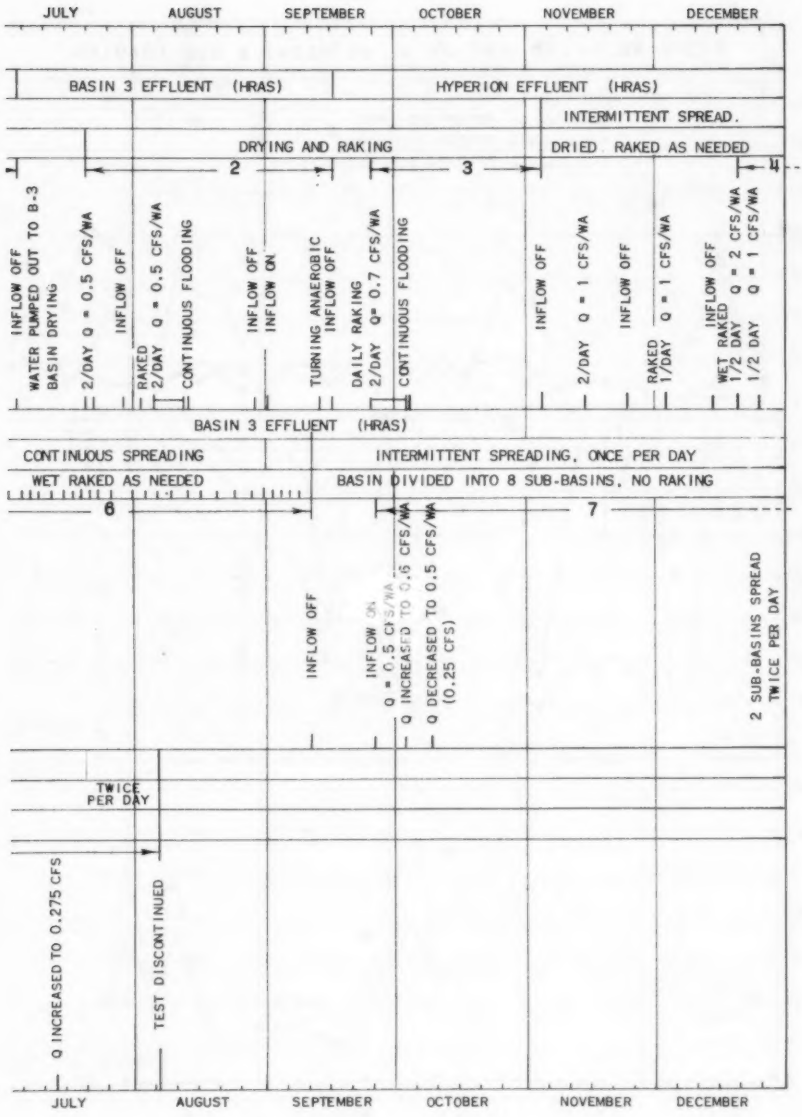


FIG. 3.—BASIN 1 CHRONOLOGY



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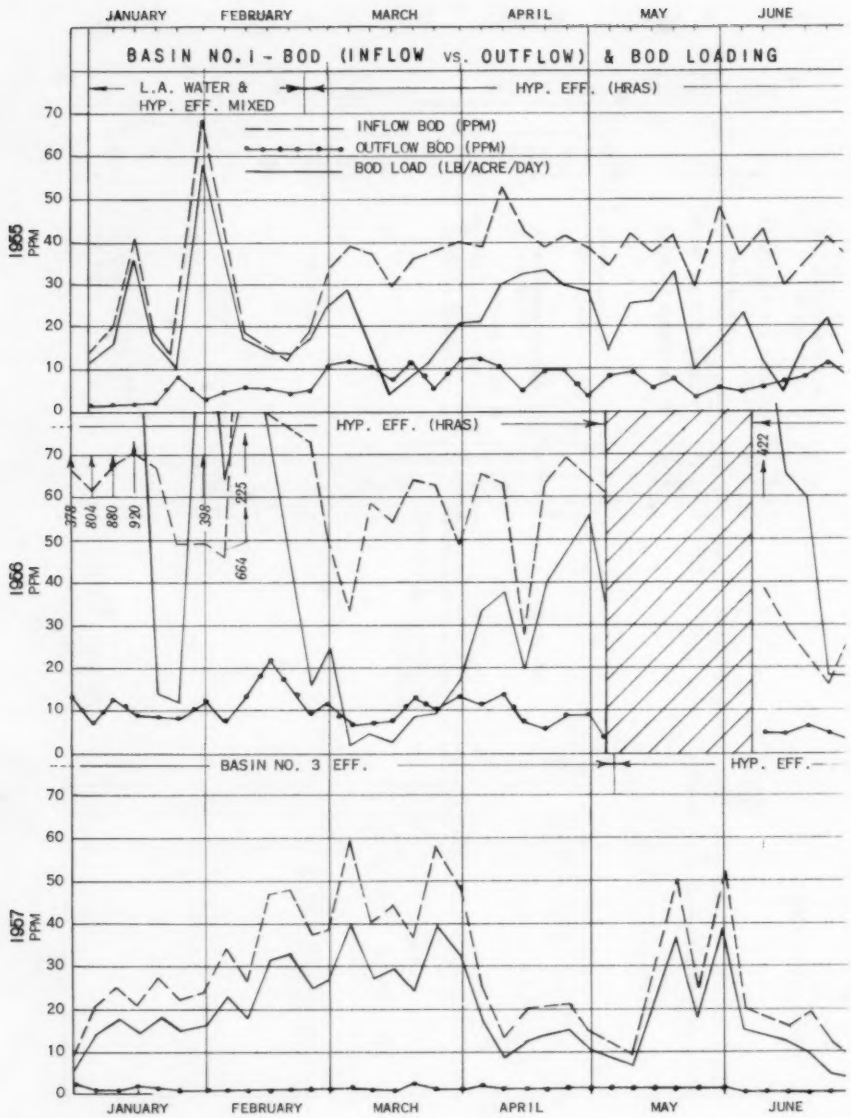


FIG. 4.—BASIN 1, 5-DAY BOD LOADING RATES

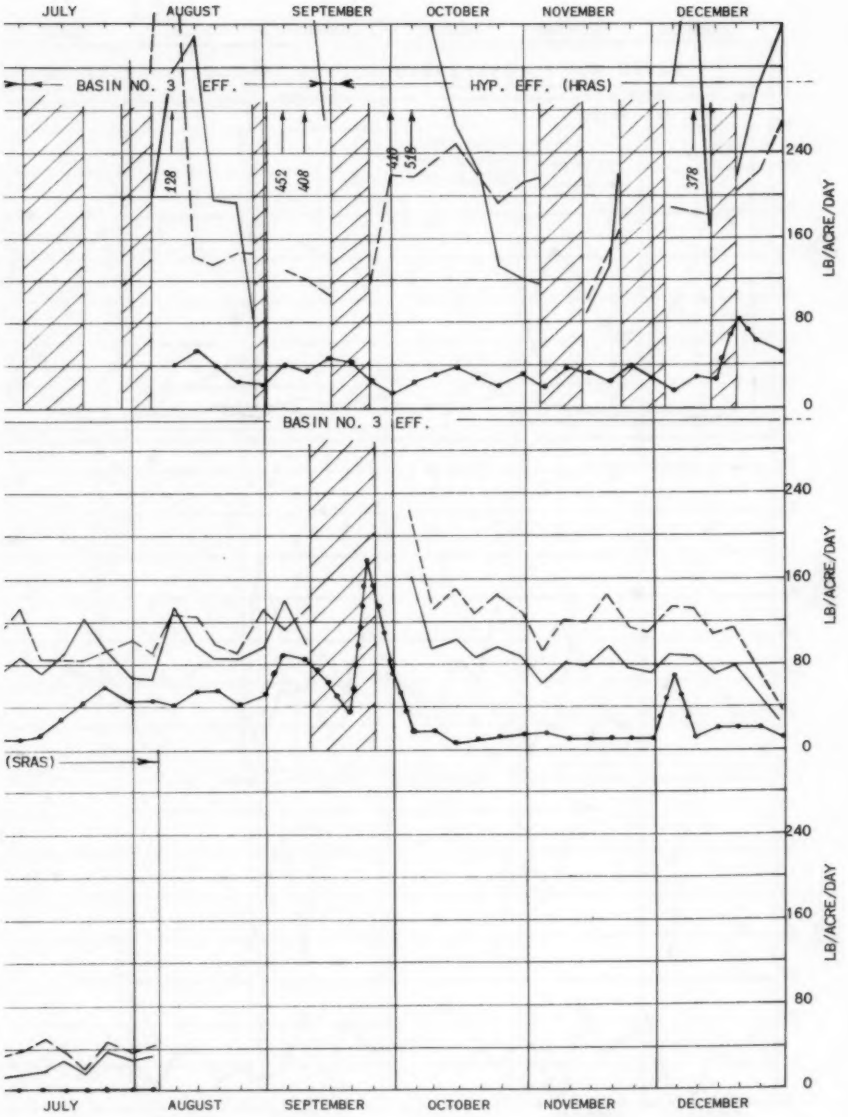


FIG. 4.—*CONTINUED*

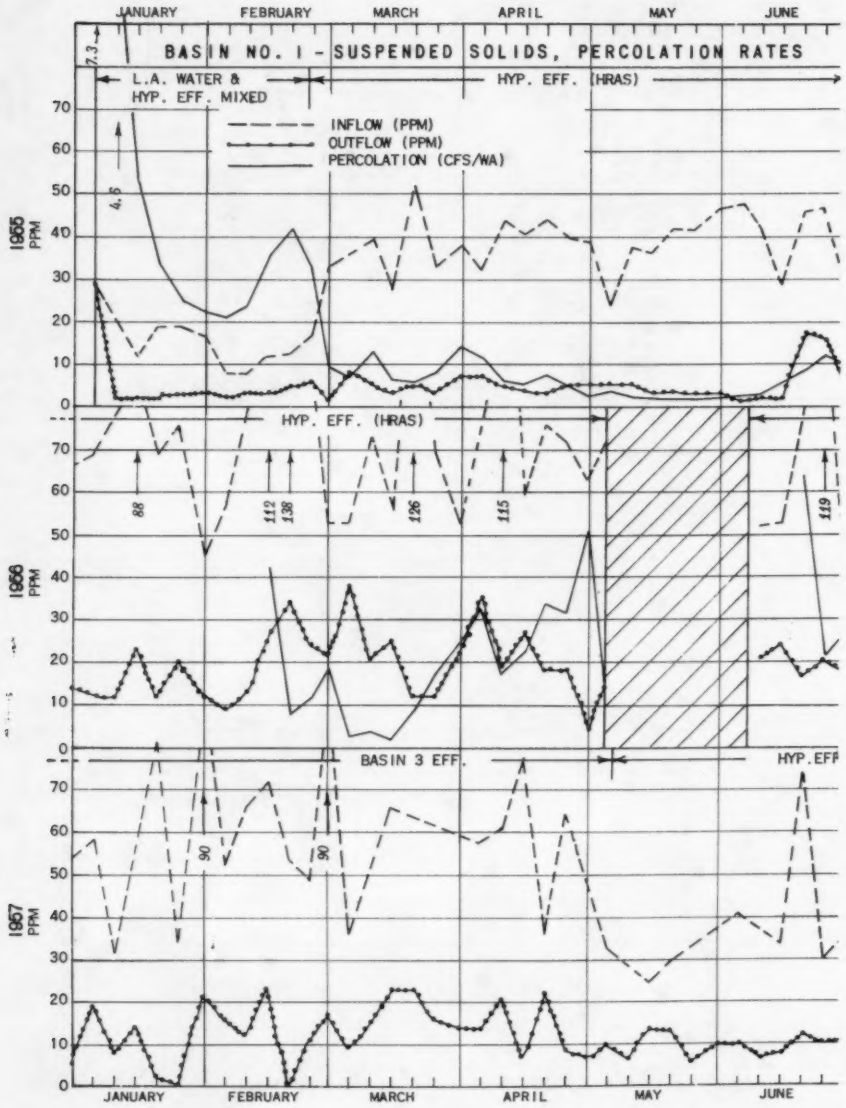


FIG. 5.—BASIN 1 - SUSPENDED SOLIDS, PERCOLATION RATES

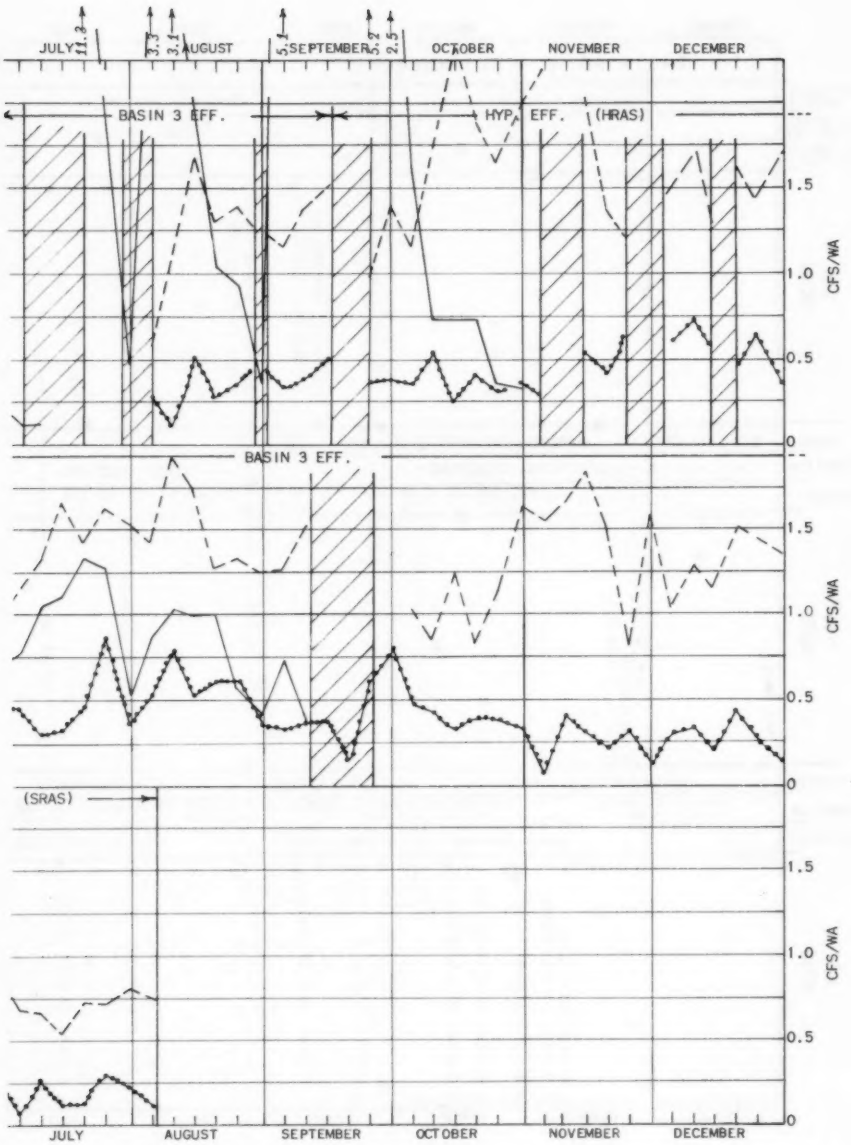


FIG. 5.—*CONTINUED*

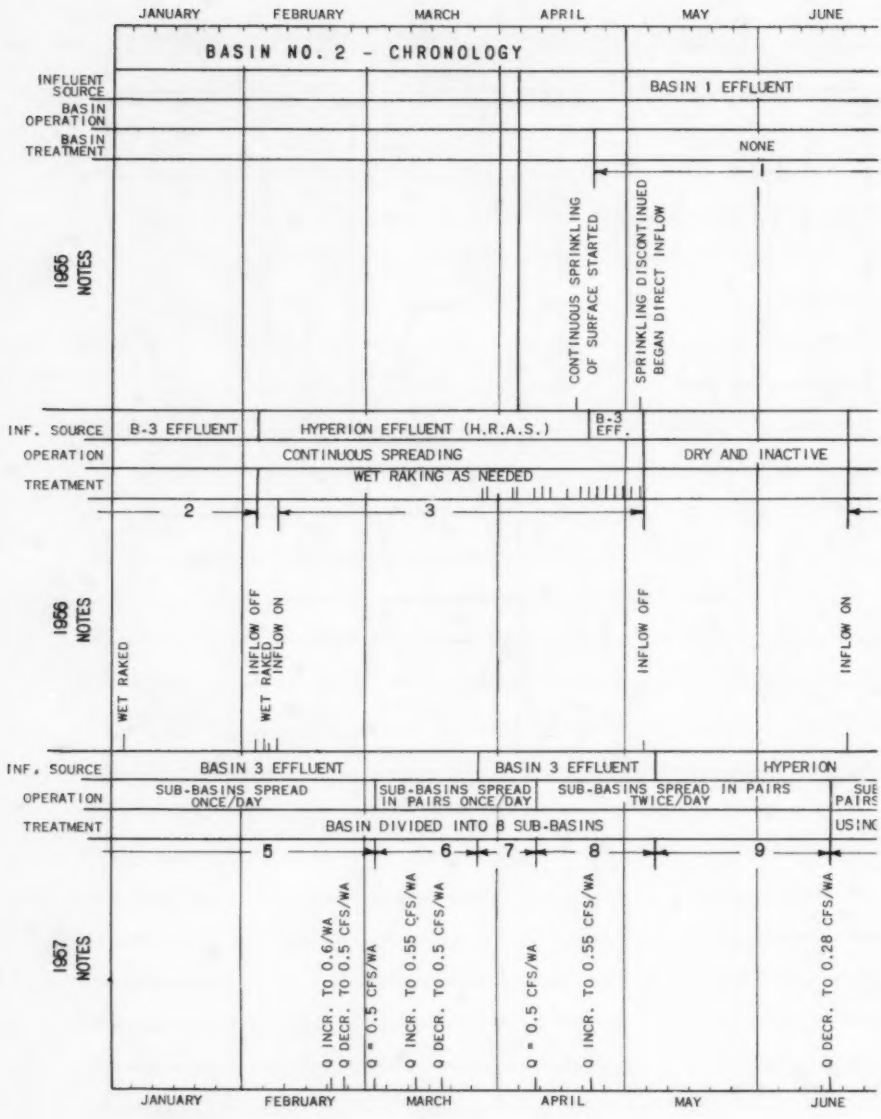


FIG. 6.—BASIN 2 CHRONOLOGY

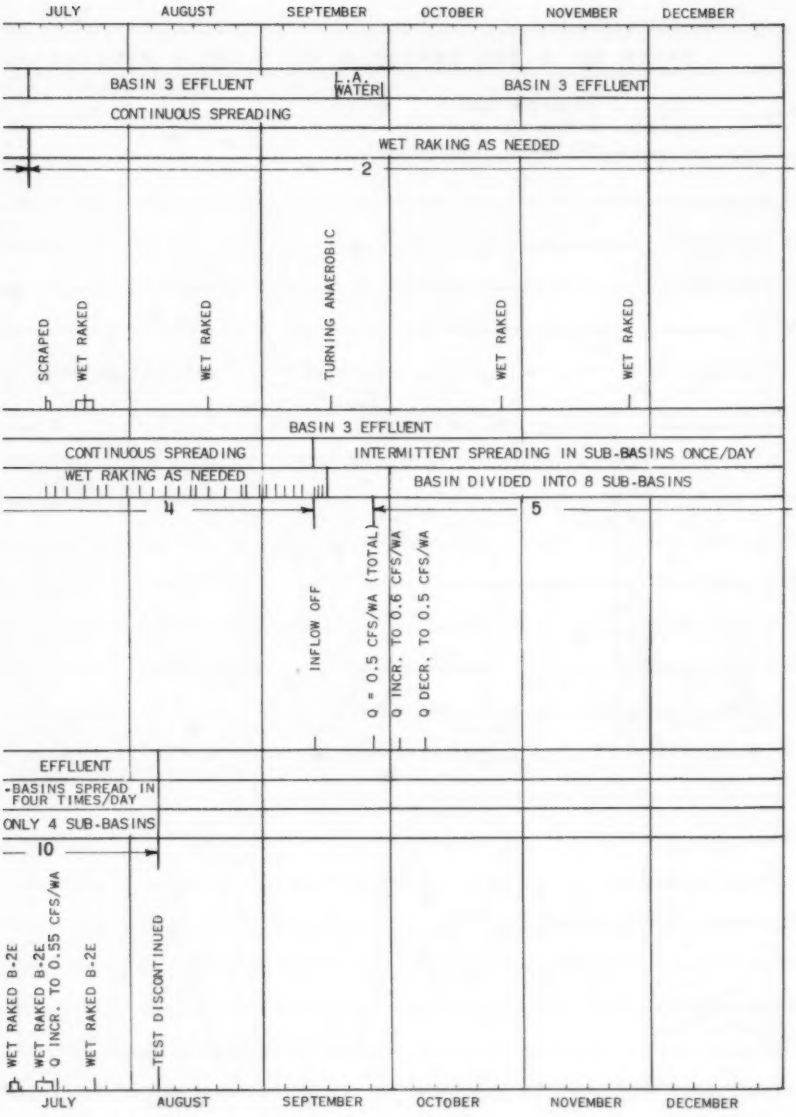


FIG. 6.—*CONTINUED*

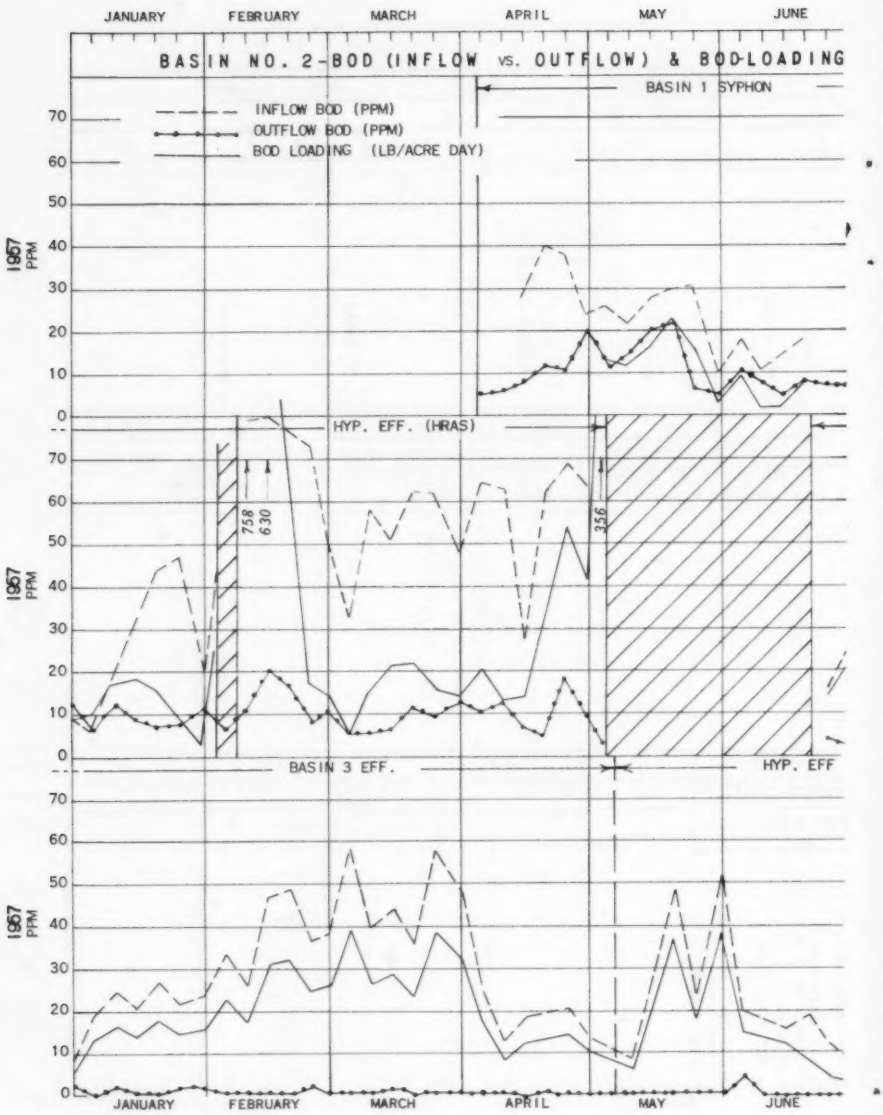


FIG. 7.—BASIN 2, 5-DAY BOD, LOADING RATES

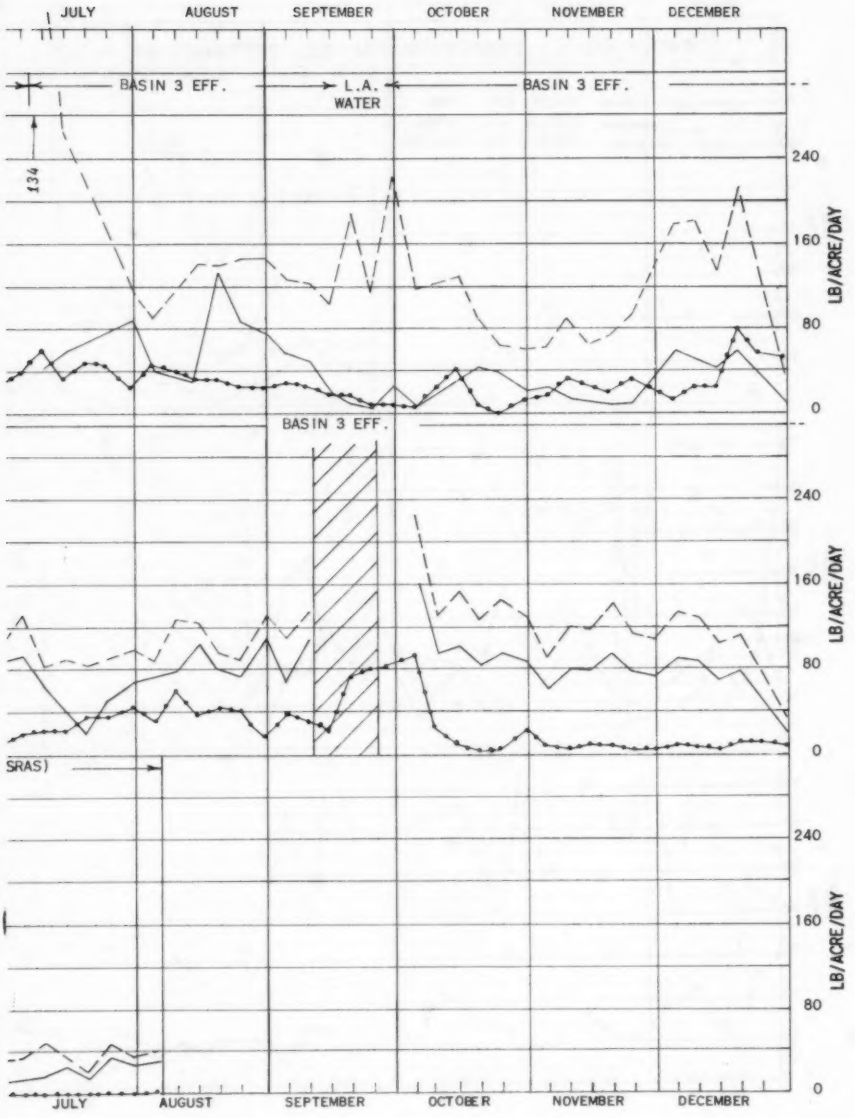


FIG. 7.—*CONTINUED*

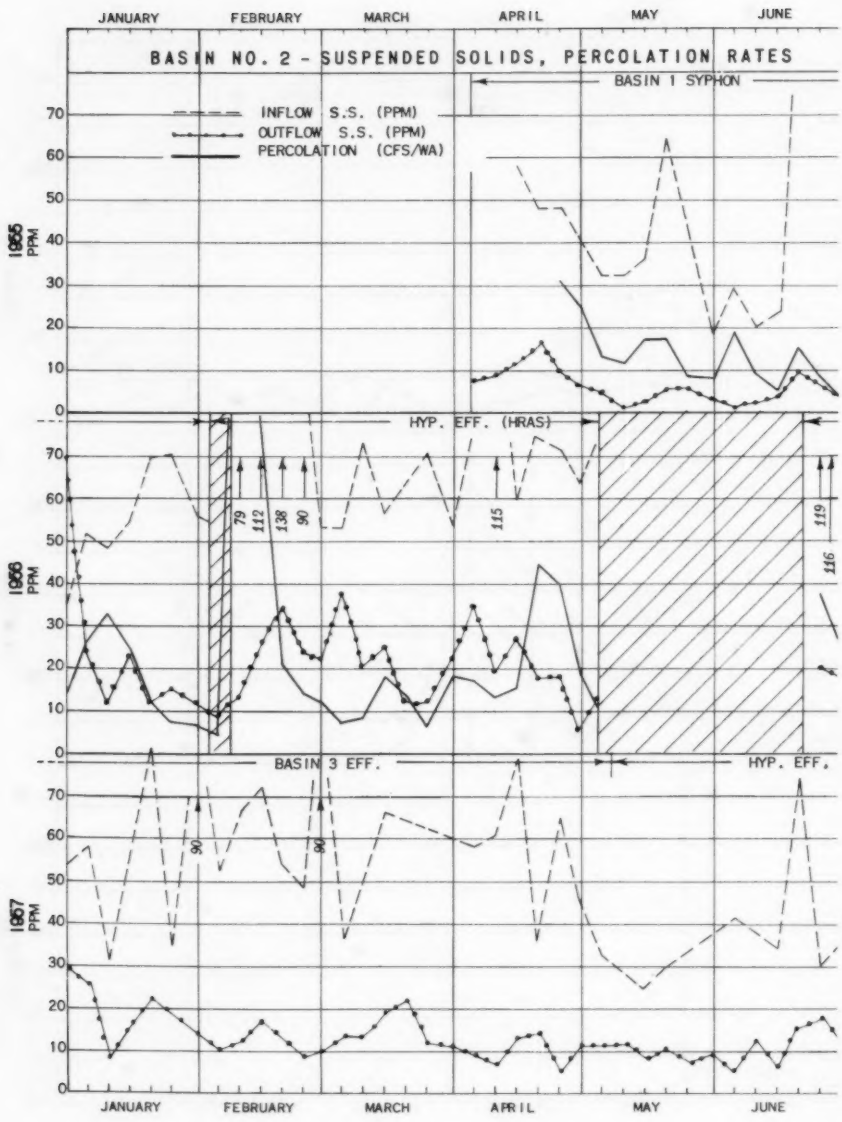


FIG. 8.—BASIN 2 - SUSPENDED SOLIDS, PERCOLATION RATES

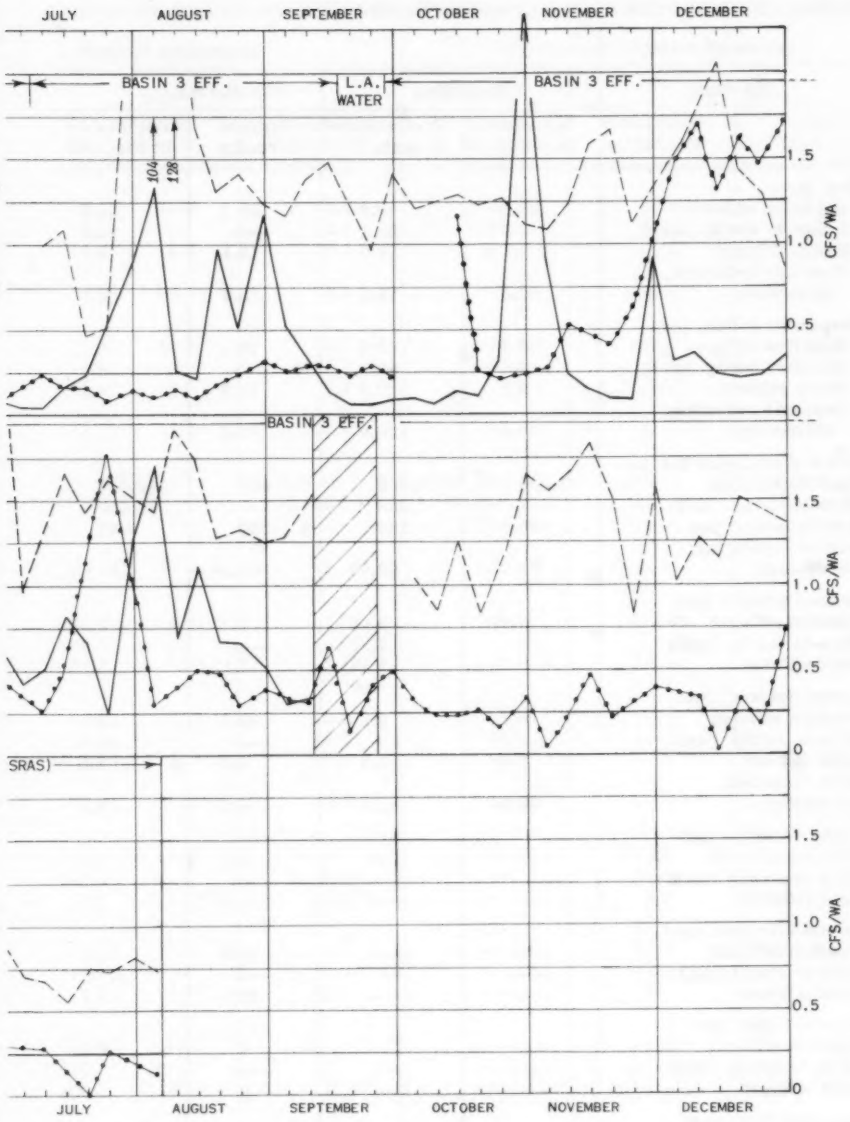


FIG. 8.—*CONTINUED*

TABLE 3.—COMPARISON OF

	Continuous Spreading			
	No Raking		Wet Raking	
	Applied Directly	Pretreated in basin #1	Applied Directly	Pretreated in basin #3
Bod, ppm				
Hyperion effluent	38.1	40.9	59.2	51.4
Inflow to sprdg. basin	----	21.4	----	28.3
Basin effluent	7.4	10.8	12.1	9.1
Over-all reduction, percentage	80.6	73.6	79.6	82.3
Suspended solids, ppm				
Hyperion effluent	33.2	40.8	79.1	50.5
Inflow to sprdg. basin	----	42.1	----	51.9
Basin effluent	0.0	0.0	17.0	14.3
Over-all reduction, percentage	100.0	100.0	78.5	71.7
Coliform count, mpn per ml				
Hyperion effluent	70,000	70,000	240,000	240,000
Inflow to sprdg. basin	----	7,000	----	62,000
*7-ft collection pan	160	160	230	230
Over-all reduction, percentage	99.77	99.77	99.90	99.90
Dissolved oxygen, ppm				
Hyperion effluent	3.80	3.80	1.35	1.59
Inflow to sprdg. basin	----	7.85	----	2.20
Basin effluent	0.55	1.16	1.12	3.63
Chlorine demand, ppm				
Hyperion effluent	9.9	9.0	19.8	18.6
Inflow to sprdg. basin	----	----	----	25.6
Basin effluent	1.0	1.3	5.1	2.2
Basin reduction, percentage	89.9	85.6	74.2	88.2
Organic nitrogen, ppm				
Hyperion effluent	----	----	7.3	7.1
Inflow to sprdg. basin	----	----	----	6.18
Basin effluent	----	----	1.4	1.2
Ammonia nitrogen, ppm				
Hyperion effluent	----	----	16.5	15.5
Inflow to sprdg. basin	----	----	----	18.1
Basin effluent	----	----	18.5	9.5
Nitrite nitrogen, ppm				
Hyperion effluent	----	----	0	0.05
Inflow to sprdg. basin	----	----	----	.037
Basin effluent	----	----	0	.05
Nitrate nitrogen, ppm				
Hyperion effluent	----	----	0.8	0.23
Inflow to sprdg. basin	----	----	----	.20
Basin effluent	----	----	5.2	6.2

[a] Values represent 7-ft collection pan samples instead of basin effluent. The under-basin effluent would show effect of such chlorination. In the case of nitrogen samples,

BASIN SPREADING METHODS

(HRAS)		Intermittent Segment Spreading		
Dry Raking		HRAS Eff.	SRAS Eff.	
Applied Directly	Pretreated in basin #3	Pretreated in basin #3	Pretreated in basin #3	Applied Directly
53.9	49.2	40.0	22.8	11.8
----	31.7	33.7	17.4	----
5.3	10.3	3.5	1.0	0.30
90.2	79.1	91.2	95.4	97.5
81.4	78.4	71.9	63.0	25.2
----	67.2	49.6	47.5	----
8.8	5.1	7.2	5.7	2.5
89.2	93.5	90.0	91.0	90.0
70,000	70,000	240,000	6,200	2,300
----	70,000	62,000	2,300	2,300
70	700	230	----	45
99.90	99.00	99.90	----	98.04
1.55	1.08	0.13	0.46	1.20
----	5.06	.03	.05	----
0.58	1.12	3.42	3.23	2.97
2.02	12.6	13.3	----	4.5
----	16.2	25.6	----	----
9.8	8.1	2.5	----	2.4
51.5	35.7	81.2	----	46.7
8.55	----	----	3.6	2.32
----	6.15	----	----	----
2.75[a]	2.28[a]	----	1.1	2.13
17.5	----	----	27.9	4.97
----	15.2	----	----	----
8.7[a]	16.6[a]	----	0	0.77
0.05	----	----	0.02	0.61
----	0	----	----	----
0[a]	0.12[a]	----	0	0.27
0.2	----	----	0.1	14.2
----	0.1	----	----	----
18.1[a]	7.3[a]	----	28.4	22.4

drains were chlorinated by means of effluent recirculation. Coliform numbers of the 7-ft pan results are shown where final effluent data is not available.

copper sulfate in concentrations up to 2 ppm likewise had no effect. Treatment with massive dosages of chlorine (200 ppm) increased the rate from 0.34 cfs per wa to 2.3 cfs per wa and again the following week, from 0.42 cfs per wa to 1.6 cfs per wa, but the improvement was transitory and the unit cost too great to warrant serious consideration.

It was noted that flow rates through the lysimeters were always correspondingly greater than in the spreading basins, presumably because of exaggerated boundary effects. Nevertheless, the results were qualitatively useful.

Periodic removal of the organic mat by mechanical means was next studied, the two principal variations being wet and dry raking. Wet raking consisted of surface scarification at intervals while maintaining continuous input. Shallow wet raking was first attempted, following which a tractor-drawn harrow was used for deeper raking and gave more lasting improvement in flow rates. The chief disadvantage of wet raking lay in the sudden marked deterioration of filtrate quality that accompanied each disturbance of the basin surface, as indicated by relatively high suspended solids and BOD concentrations. Also, it was found necessary to increase the wet raking work from once in several weeks to as often as once each day. Sample cores of the spreading basins subsoils indicated that the subsurface soils were black, odoriferous, and anaerobic. It is probable that this anaerobic environment was chiefly responsible for the relatively inefficient removal of suspended solids and soluble organic matter.

It was at this point in the test that the land counted on for spreading was lost because of airport development, and it was realized that a satisfactory high-rate tertiary treatment had to be developed to accomplish well injection without spreading. The first step was to construct a recharge well (described subsequently), and the second step was to test the well with the effluent then available from the raking operations. Figs. 9 and 10 depict the recharge well operational parameters and test results, as well as the chronology of operations. Thus, Fig. 9 indicates that Well Runs 1 and 2 were made July 5 through 25, 1956, and August 1 through September 10, 1956, respectively; and that the wet raking effluent caused excessive well head increase; consequently, this was an unsatisfactory method of reclaiming Hyperion effluent.

In dry raking, the basin was taken out of service whenever the cumulative average percolation rate dropped below 1 cfs per wa, usually at intervals of about 2 weeks. The basin was then drained, dried thoroughly and harrowed. Because the down time for draining and drying was generally about 8 days, the efficiency of this technique was somewhat low. However, the quality was almost as good as that achieved with no raking. The use of effluent from the oxidation pond did not appreciably change the results obtained by raking. Raking operation data are shown in Table 3.

During the dry raking studies it was noted that each time the basin was drained a black, mucky, anaerobic mat 2 in. to 3 in. thick appeared, which oxidized rapidly on exposure to sun and air. Within a day or two, in fact, it would disappear completely, leaving a surface layer of clean sand. Based on this observation, a series of lysimeter tests was run using intermittent spreading consisting of cyclical application and drainage. As the sand drained it was rapidly aerated and formation of this sulfide type of mat did not occur.

Prototype intermittent spreading was initiated in Basin No. 1. A depth of 1 ft of sewage effluent was applied at 12-hr intervals (equivalent to 1 cfs per wa). Hydraulic failure soon occurred; that is, the basin had not drained by the time the next sewage application was due. A single application of 2 ft once

per day was likewise unsuccessful. A schedule of one dosing every second day was adopted to allow a longer period between applications for drying and re-aeration of the basin soil. This run was somewhat better, but it also failed in a few days and had to be supported by auxiliary raking. It appeared that an average flow rate of 1 cfs per wa could not be maintained because the time required to fill the basin with sewage effluent was so great, using the two available 1,000 gpm pumps, that there was not sufficient time left for proper draining, drying, and re-aeration.

Because the objective at this stage of the test was to obtain 0.5 cfs continuous flow for the well recharge test, each of the half-acre basins were divided into eight sub-basin areas. One foot of waste water was pumped into each sub-basin daily during a half-hour period, whereas the remainder of the daily operating cycle was used for drying and aeration to preserve the spreading basin percolation characteristics. This method was entirely successful. At no time was auxiliary raking required, nor did the drainage capacity of the basin soils diminish during the 11 months of operation. The percolated waters obtained were sparkling clear, odor-free, and of superior quality as compared to those derived from continuous spreading, with or without raking. Core samples from the test spreading basins indicated that the sulfide content of the underlying sand had been entirely eliminated and its natural color and appearance restored.

The results of reclamation by intermittent sub-basin or segment spreading are shown in the final three columns of Table 3. Clearly, this polishing technique yielded water significantly higher in quality than the other spreading methods studied. Moreover, when a more highly treated sewage (SRAS effluent) was reclaimed, water of excellent quality was obtained.

Well Runs 3 and 4 demonstrated improved reclaimed water quality. The thoroughly successful Well Run 5 also utilized water reclaimed by intermittent sub-basin spreading and will subsequently be described more completely.

It is interesting to note from Table 3 that, although pretreatment by ponding effected a partial reduction in the BOD and suspended solids content of the Hyperion effluent, as indicated in Table 2, the improvement did not carry over to the reclaimed water.

Spreading Basins Soils.—Mechanical sieve analyses of the test spreading basin soils were performed on core samples prior to and following 17 months of the previously described spreading operations. The results of these sieve analyses indicated no significant change in the mechanical properties of the basin soils. On completion of all waste water percolation tests, the permeability characteristics of the spreading basins were found to be essentially unchanged.

HIGH RATE RECLAMATION

Several test units were designed and constructed in order to determine whether reclamation could be accomplished at significantly higher rates than those found feasible by spreading. Among these units were a trickling filter combined with an upward-flow sludge blanket clarifier and several sand and gravel filters. These were generally tested on a pilot-plant scale, using lysimeters. However, a pea gravel filter, incorporating automatic flow control and raking and followed by a final clarifier, operating at an equivalent rate of

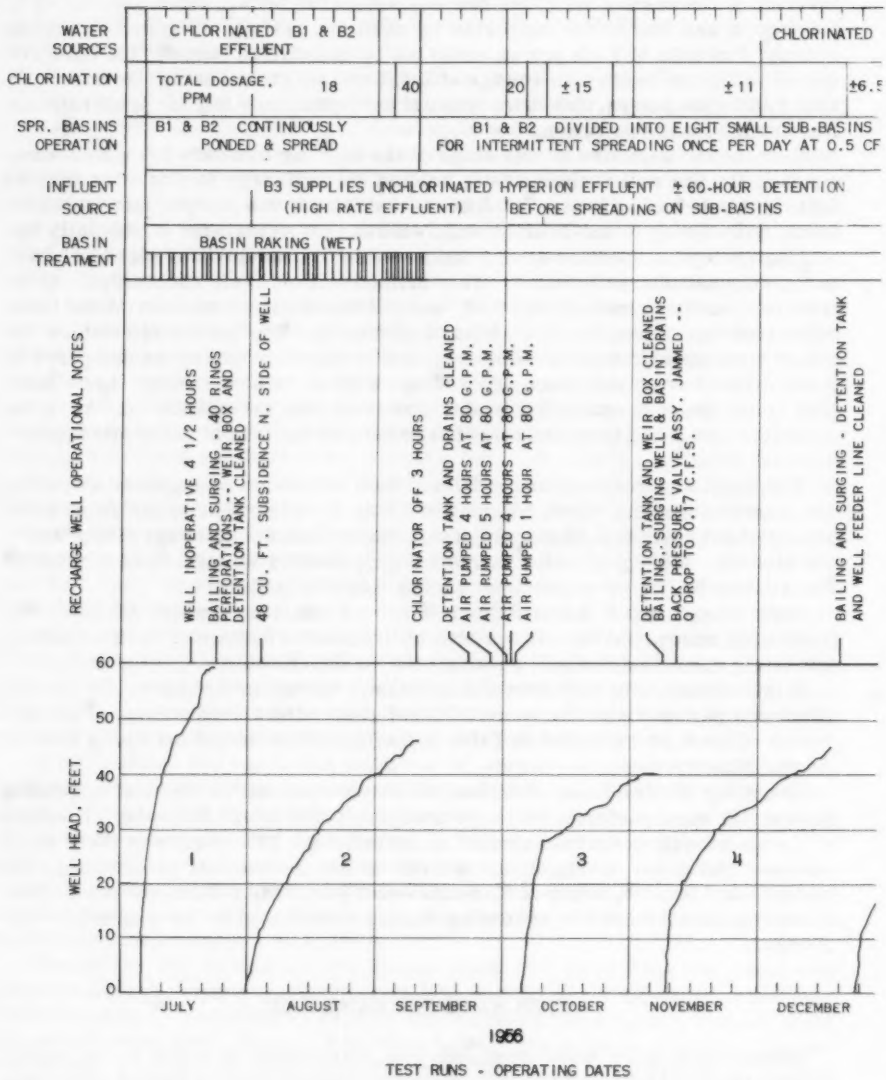
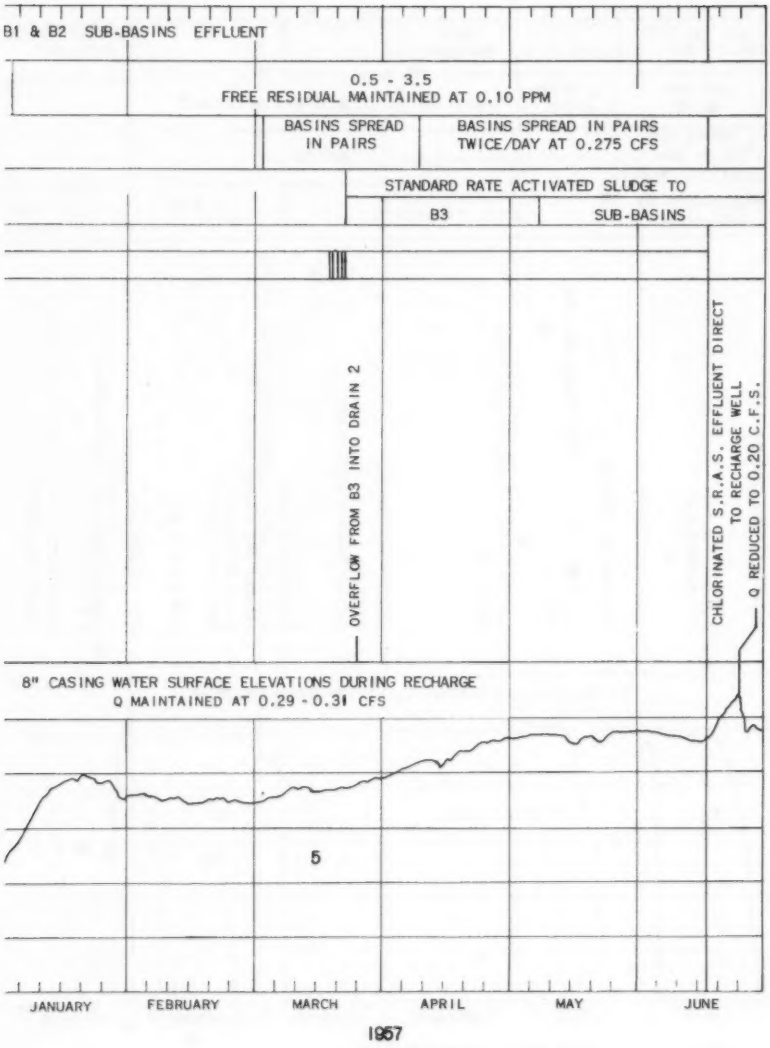


FIG. 9.—RECHARGE OPERATIONS - WELL RUNS NOS. 1-5



TEST RUNS - OPERATING DATES

FIG. 9.—*CONTINUED*

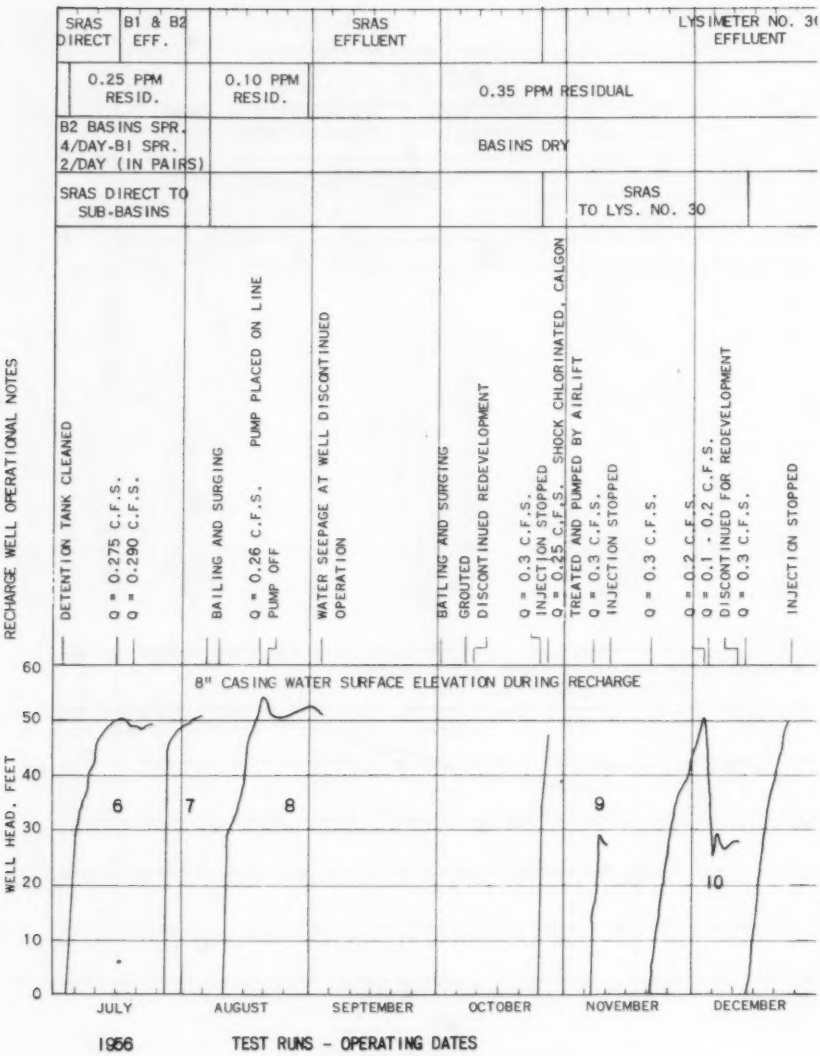


FIG. 10.—RECHARGE OPERATIONS - WELL RUNS NOS. 6-10

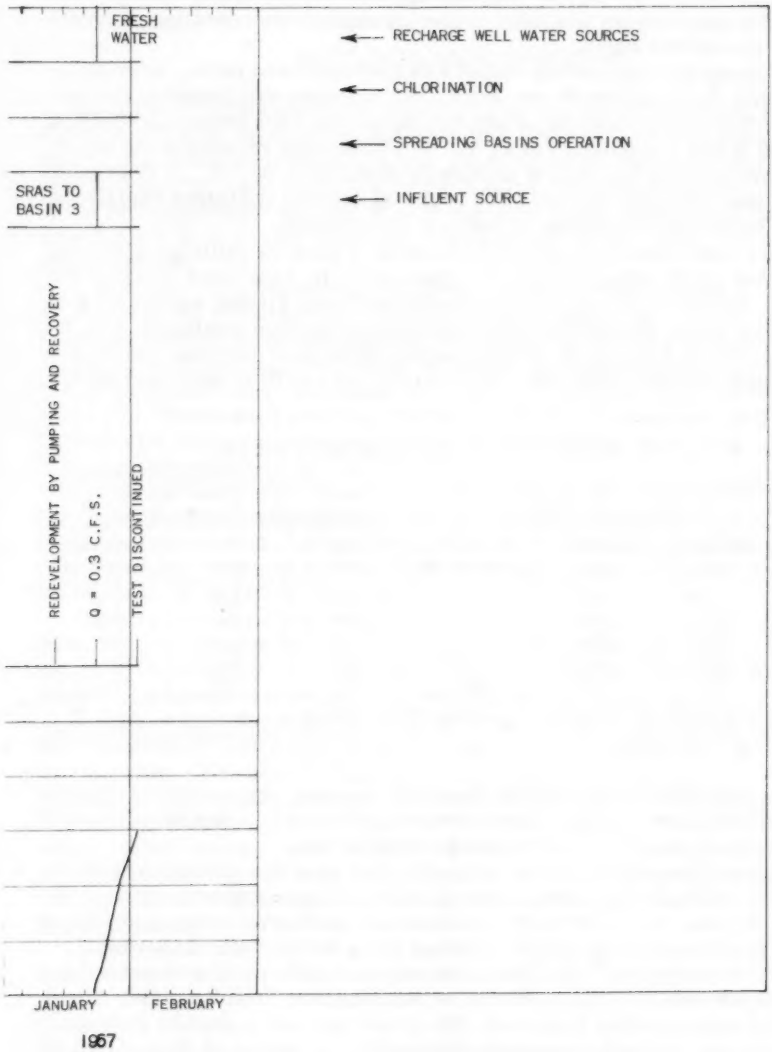


FIG. 10.—*CONTINUED*

24 cfs per wa, was built to a size (30 ft diameter) sufficient to supply 0.3 cfs to the experiment recharge well. Although high flow rates were obtained from most of these units, the degree of polishing was not generally satisfactory. Sufficient test time was not available to further explore the operating possibilities of the pea-gravel filter.

A promising high-rate method tested was the rapid sand filter. A proprietary type pilot filter equipped with automatic backwash was loaned to the district. The filter was divided into eight compartments, each having an effective area of 1 sq ft and a sand depth of 11 in. The filter sand recommended had an effective size of 0.95 mm and a uniformity coefficient of 1.6. A finer sand (0.45 mm) was also used but had to be replaced because it clogged rapidly and caused the backwash mechanism to operate continuously.

The rapid sand filter was operated for about 6 months, utilizing SRAS effluent applied at an optimum rate of 2 gpm per sq ft, equivalent to about 200 cfs per wa. No chemical treatment, to flocculate finely divided suspended solids remaining in the effluent following conventional sewage treatment, was attempted. Perhaps because of clogging of the filter sand by these solids, and partly because the backwash cylce of 1 min did not result in sufficient expan-

TABLE 4.—PERFORMANCE OF RAPID SAND FILTER

Run No.	Suspended Solids			Biochemical Oxygen Demand		
	Inflow,[a] ppm	Filtrate, ppm	Removal, percentage	Inflow,[a] ppm	Filtrate, ppm	Removal, percentage
1	18.5	6.5	64.9	5.8	2.4	58.6
2	28.1	12.6	55.2	6.9	2.8	59.4
3	27.3	13.5	50.5	10.3	2.7	73.8
4	34.0	19.9	35.3	15.2	5.3	65.2
5	27.7	21.0	24.2	11.0	7.8	29.1

[a] The influent to the filter during each run was SRAS effluent.

sion of the sand bed to accomplish thorough cleaning, the quality of filtrate gradually deteriorated. This is demonstrated in Table 4 in which the suspended solids removal is shown for each successive filter run.

Chlorination Procedures.—Throughout the test program shock chlorination, involving the application of intermittent dosages as high as 40 ppm chlorine for half-hour periods, was found to be a satisfactory method of keeping pipes and other test facilities free of bacterial slimes and other biological growths.

The test recharge well (HR Well) was supplied with reclaimed water continuously dosed with 0.5 ppm to 40 ppm of chlorine (see Figs. 9 and 10). Gaseous chlorine was supplied from both 150-lb and one ton cylinders through a conventional type manually operated chlorinator. A review of Figs. 9 and 10 indicates that probably the most successful chlorination procedure for well injection occurred while operating at a dosage ranging from 0.5 ppm to 3.5 ppm in order to maintain a free chlorine residual at the recharge well of about 0.1 ppm chlorine.

High chlorine residuals (5 ppm - 10 ppm) during the initial recharge well tests appear to have caused metallic corrosion and fouling of the well by fer-

ric hydroxide. The test data indicate that 0.1 ppm residuals were satisfactory both in maintaining recharge well permeability and in reducing coliform counts to acceptable levels.

RECHARGE INJECTION TEST

Injection of reclaimed waste water into the recharge well (HR) constituted the second major phase of the Hyperion test program. An 8-in. diameter steel casing was placed to a depth of some 140 ft below ground surface and perforated between 98 ft and 132 ft within the confined Silverado aquifer. The latter is the principal water-bearing zone of the West Coast Basin, but it thins to approximately 50 ft at this location. The well was packed with a 2-in. to 4-in. gravel blanket.

A series of three observation wells was drilled at distances of 20 ft and 70 ft northerly of the recharge well and parallel to the shore line, and 50 ft inland of the recharge well in the direction of the normal hydraulic gradient. The 8-in. casing of each well was perforated within the confined aquifer. Unfortunately, there was no confining clay cap at the location of a fourth well, 500 ft from HR, which had been drilled earlier. Surface seepage into this well prevented obtaining valid test data on lateral flow from the recharge well. The well system is shown schematically in Fig. 2.

Reclaimed water was supplied to the recharge well by gravity flow under sufficient head to accomplish injection at a rate of 0.3 cfs against a maximum pressure head of 50 ft. A back-pressure valve in the injection line controlled the rate of flow and kept the line filled with liquid, preventing air-locks due to introduction of entrained air.

Prior to recharge the reclaimed effluent was chlorinated, as previously described, and passed through a contact chamber having a 40-min contact period, in order to achieve disinfection of B. coli, sulfur bacteria and other slime-producing bacteria that might otherwise foul the well.

Bi-hourly samples were taken at the recharge well tap during injection for determination of chlorine residuals. Composites were prepared for daily chemical analysis. The observation wells were sampled at weekly intervals after pumping to equilibrium, as shown by electrical conductivity readings.

Twelve separate well runs were made. Each run was terminated when the recharge well injection head exceeded 50 ft and reduced the flow below 0.3 cfs, following which the well was redeveloped by surging and bailing with a cable-tool rig. Later a high-capacity (600 gpm) airlift pump was used in an effort to obtain more effective redevelopment.

On December 24, 1956 (see Fig. 9), Well Run 5 was started, and intermittent sand filtered water was accepted continuously by the test well for a period of over 6 months with minimal injection head increase. The hydraulic characteristics of the recharge well during this run resembled those encountered in the successful operation, since 1953, of recharge wells with filtered and softened Colorado River water at the West Coast Basin Barrier project. The reclaimed water quality obtained by means of intermittent spreading appeared to be satisfactory for continuous operation of a recharge well system. The recharge test was thus demonstrated during Well Run No. 5 to be thoroughly successful. Pertinent information on the first six well runs is presented in Table 5.

During Well Run No. 7, the use of proprietary chemicals was investigated as part of an attempt to improve coagulation and removal of suspended solids during the SRAS treatment operations. However, carry-over of bentonite, used in this investigation, into the injection water, resulted in complete fouling of the well.

TABLE 5.—RECHARGE WELL OPERATION

Well Run No.	Duration, days	Water Source	Chlorine dosage, ppm	Type of basin polishing	Clogging[a] rate, in feet per day	Remarks
1	18	Basin-polished HRAS	18	Continuous spraying of B-3 eff with wet raking.	1.75	Final inj. head 59.5 ft. Casing reperforated.
2	42	Ditto	18-40	Ditto	0.61	When Cl_2 was increased to 40 ppm, inj. head rose sharply.
3	33	Ditto	11-20	Intermittent spraying in sub-basins, one per day.	0.42	Run ended by jamming of back pressure valve assembly.
4	39	Ditto	11 (5 ppm residual)	Intermittent spraying in sub-basins, one per day.	0.59	
5	183	Ditto (see remarks)	0.5-3.5 (0.1 ppm residual)	Intermittent spraying in sub-basins, 2 cycles per day.	0.065	Direct recharge of SRAS fouled well, ended run.
6	21	Direct SRAS, then basin percolate	0.25 ppm residual	Intermittent spraying in sub-basins, 2 cycles per day.	0.89	Inj. head leveled off when using polished water.

[a] See subsequent section of report for discussion of clogging rate.

Clogging Rate.—During operation of a reverse well, the back-pressure is inversely proportional to the well's absorptive capacity. Even when clear water is injected, a characteristic period of rapid increase in the injection head usually occurs. After a few days the head becomes more or less constant and then may actually show a slight temporary decrease. Any subsequent increase may be attributed to clogging, due to one or more of the following causes:

1. Deposition of suspended matter within the interstices of the soil surrounding the well;
2. Development of slime organisms at the casing interface, utilizing nutrients in the injected water;

3. Injection of an incompatible or scale-forming water causing respectively, precipitation of insoluble compounds and cementation of sand grains, plugging of casing perforations, or sealing of gravel envelope; and

4. Decrease in aquifer permeability due to peptizing of fines by surface exchange principally of calcium by sodium ions.

The saturation index of the reclaimed waste water used in this investigation was very slightly negative, indicating a non-scale-forming water. This was further corroborated by metal corrosion-test strips. The sodium equivalent ration was near the upper limit and averaged 60%. Moreover, direct laboratory tests showed no tendency toward precipitation of solids when this water was mixed with ground water samples. Likewise, underground biological growth was inhibited by the chlorine added prior to injection. Deposition of suspended solids, possibly including bacterial bodies in the recharge water must, consequently, be a major factor contributing to injection well clogging.

During the experimental injection test well operation, the injection head typically rose quickly to 35 ft or more, regardless of the quality of the reclaimed water. Out of a total practicable recharge head of about 50 ft, above which the nominal flow rate of 0.3 cfs could not be maintained, the usable head increase, therefore, was only about 15 ft. In order to avoid costs of taking a well out of service for redevelopment more than twice per year, the clogging rate should be less than 15 ft in 180 days, or 0.083 ft per day.

The average clogging rate while Well Run No. 5 was fed by effluent from intermittent spreading was only 0.062 ft per day. During this run the suspended solids content of the reclaimed sewage averaged 6.3 ppm with a daily range from 0 to 30 ppm. The average BOD during this run was 1.2 ppm.

Based on these results, apparently a solids content of about 6 ppm can be tolerated under Hyperion Test conditions without unduly increasing the clogging rate. On direct injection of SRAS effluent having a suspended solids concentration of 25 ppm, however, the clogging rate rose immediately to more than 1 ft per day, necessitating termination of the test run. It is important to note, of course, that the effect of suspended solids on clogging is a function of the biochemical nature, structure, and particle size of the solids as well as their concentration.

Inasmuch as a sewage plant, even one utilizing so-called "complete" secondary treatment, normally produces a final effluent containing more than 6 ppm suspended solids, further tertiary treatment will generally be required prior to injection.

Injection Water Quality.—The bacterial, organic, physical, and chemical quality of the injected water is depicted in Table 6. This table also reviews the recommended standards of water quality for domestic and agricultural uses. It can be seen that the reclaimed water is acceptable by recognized standards.

Waste-water mineralization probably can be best controlled to conform to these standards by segregation and elimination of undesirable or highly mineralized wastes from the sewage system. New construction by the city of Los Angeles has been completed since the test for bypassing the highly mineralized waste waters from the Venice, Calif. area as well as controlling other industrial wastes now entering the municipal sewerage system. This reduction in saline brines and industrial waste pollution is an important step toward protecting the mineral quality of the Hyperion plant effluent.

TABLE 6.—EVALUATION OF WATER QUALITY (DATA IN PPM EXCEPT AS NOTED)

Analysis	Recommended Limiting Standards		USDA[a] Irrigation Water	Rep. Injection Well Water (Well Run 5)
	USPHS Potable Water	Calif. State Health Dept.		
Bacteria				
Coliform mpn/ml	0.22	0.22	100	.022
Organic				
BOD (5 day 20° C)	—	—	—	1.2
Ether Solubles	—	—	5.0	1.0
Volatile Acids	—	—	—	7.8
Physical				
pH	—	—	6.0 to 9.0	7.1
Color	20	20	—	9[d]
Suspended Solids	—	—	—	6.3
Odor	Minimum	Minimum	Noxious	Negligible
Taste	Minimum	Minimum	—	Negligible
Turbidity as SiO_2	—	—	—	—
Chemical				
Calcium, Ca	—	—	—	77
Magnesium, Mg	—	150[b]	—	28
Sodium, Na	—	—	—	213
Iron & Manganese				
Fe & Mn	0.3	0.3	—	—
Chlorides, Cl	250	600[b]	500	223
Sulfates, SO_4	250	600[b]	—	154
Ammonia, NH_3	—	—	—	1.8
Nitrites, NO_2	—	—	—	0.03
Nitrates, NO_3	—	44	—	94.3
Fluorides	1.5	.8-1.5[c]	—	—
Dissolved Solids	1000	1500[b]	3200	1050
Volatile Suspended Solids	—	—	—	4.9
Hardness as $CaCO_3$	—	—	—	305
Boron, B	—	—	4	0.8
Phenols	0.001	0.001	—	0.001-002
Toxic Metals	0.05	0.05	—	—
Detergents, ABS	—	—	—	1.12

[a] See Limitations and Definitions in USDA Salinity Laboratory, Circular #969.
[b] Temporary Permit, upper limits.
[c] Varies with mean annual air temperature, upper limits of solids content.
[d] Average of Well Runs 6-10. No determinations during Run 5.

Effect of Ground Water Travel on Reclaimed Effluent.—In the Hyperion test a determination was made of the effect of percolation on nitrate and other consitituents of the reclaimed water. The ground water in this case has been highly degraded by salt-water intrusion. Table 7 gives a comparison of ground water quality at the test site before and during injection, following establishment of equilibrium during Well Run 5.

Chloride was selected as a tracer by which to measure the degree of dilution that occurred during the experimental recharge. The reasons for its choice were as follows:

1. A difference in concentration of approximately sixty-fold existed between the reclaimed Hyperion effluent and the saline ground water;

TABLE 7.—EFFECT OF GROUNDWATER TRAVEL ON NITROGEN CYCLE, DETERGENTS AND MINERAL CONSTITUENTS

	Groundwater Before Recharge	HR Well	Observation Wells		
			HR-20	HR-50	HR-70
Dilution by Groundwater, percentage		——	.02	.07	4.7
Nitrate, NO_3					
Calculated, ppm		——	94.3	94.2	89.9
Observed, ppm	0.4	94.3	89.4	80.6	77.9
Reduction, percentage		——	5	14	13
Nitrite, NO_2					
Calculated, ppm		——	.01	.01	.01
Observed, ppm	0.0	.01	.06	.21	.19
Ammonia, NH_3					
Calculated, ppm		——	1.50	1.50	1.43
Observed, ppm	0.2	1.50	1.29	1.20	0.94
Reduction, percentage		——	14	20	35
Organic Nitrogen					
Calculated, ppm		——	2.20	2.20	2.12
Observed, ppm	0.5	2.20	1.59	1.36	1.06
Reduction		——	28	38	50
Sulfate, SO_4					
Calculated, ppm		——	154.9	155.9	246.4
Observed, ppm	2,110	154.5	148.4	155.2	249.3
Reduction, percentage		——	4		-1
Detergents as ABS					
Calculated, ppm		——	1.12	1.12	1.07
Observed, ppm	——	1.12	1.04	1.03	1.02
Reduction, percentage		——	7	8	5
Constituent			Concentration		
Chloride, ppm	16,410	223.2	227.6	235.3	989.6
Calcium, ppm	1,560	77.1	73.8	79.3	124.4
Magnesium, ppm	1,070	27.7	26.8	28.2	85.0
Sodium, ppm	7,890	212.7	209.5	207.4	595.7
Bicarbonate, ppm	151	236.0	226.1	271.7	271.5
Hardness, ppm	8,310	305.4	295.7	314.6	662.5
pH	7.4	7.2	7.0	7.0	7.1
Phosphate, ppm		16.8	15.5	6.8	7.1

2. Chloride compounds are predominantly soluble; consequently, there is negligible loss due to precipitation with the aquifer soil; and

3. Chloride shows practically no tendency toward ion exchange absorption by soils. Other researchers[8] have estimated the maximum amount of such absorption to be 0.2% - 0.3%.

8 "Application of Chemical and Radioactive Tracers in Hydraulic Sciences," by W. J. Kaufman, presented at the February, 1960 Conf. on Fluids and Rocks at the Univ. of California, at Los Angeles, Calif.

The degree of dilution by ground water was calculated from the chloride concentration shown for each sampling station in Table 8 using the following formula:

$$D = \frac{100\ (Cl_e - Cl_w)}{Cl_g - Cl_w} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which D denotes the percentage dilution, Cl_g is the chloride content of ground water recharge, Cl_w refers to the chloride content of reclaimed water, and Cl_e is the equilibrium chloride content at given station. The dilution as calculated from the Eq. 1 represents the theoretical concentration of any constituent at any observation well. Determination of the actual concentration of

TABLE 8.—EFFECT OF GROUNDWATER TRAVEL ON SANITARY AND BACTERIAL WATER QUALITY

	Observation Wells			
	HR Well	HR-20	HR-50	HR-70
Dissolved Oxygen, O_2, ppm avg.	5.6	3.8	1.4	1.0
Number of samples	180	24	22	23
Range	0-7.2	0-5.4	0.3-4.6	0.4-4.6
Bacterial Plate Count (#/ml), avg.	260	2,200	1,320	1,020
Number of samples	25	26	26	23
B. Coli, MPN per 100 ml	2.2	45-[a]	45-[a]	45-[a]
Biochemical Oxygen Demand, ppm avg.	1.2	1.2	1.0	1.3
Number of samples	160	22	25	22
Range	0-6	0-2	0-3	0-2
Suspended Solids, ppm	6.3	3.3	1.4	7.5
Turbidity, ppm	10.3	9	7	11
Chlorine Demand, ppm	0	1.7	1.7	2.5

[a] The minus sign following the most probable number indicates that, at the dilutions used for planting samples, all broth culture tubes were negative.

that constituent by analysis of observation well samples will then indicate the reduction that has occurred during percolation.

It will be noted from the foregoing tables that most, if not all, of the organic constituents underwent some degree of reduction during underground travel. The calculated nitrate reduction, if extrapolated as a first-order reaction should result in a concentration falling below the recommended limit of 44 ppm (cf Table 6) within approximately 300 days. Based on a necessary recharge mound height of 15 ft and applying Darcy's Law with an assumed coefficient of soil permeability of 120 ft per day (representative of the merged Silverado aquifer in the vicinity of the recharge test), the distance traversed by the water during this period will be considerably less than 1,000 ft.

The mechanism of nitrate reduction was studied by means of special laboratory tests. It was demonstrated that considerable removal of the total nitrogen content was obtained by means of bacterial action in an anaerobic en-

vironment, probably through production of nitrogen gas as an end product. In an aerobic environment minor removal of nitrogen compounds was encountered, but at a slower rate than that occurring anaerobically.

With respect to nitrite, it is believed, in view of the small concentrations involved and the transient nature of this constituent, that its effect on the nearest permissible water supply well would be negligible.

The high rates of reduction observed for both ammonia and organic nitrogen appear to indicate that these constituents would be rapidly depleted and have little effect on the domestic water supply.

The data on sulfates indicate extremely close agreement with the chloride and corroborate the dilution equation. As might be anticipated, the sulfate shows little or no progressive change except for the increased concentration caused by the high-sulfate intruded water.

Phosphate, like sulfate, is of little public health significance. However, it is essential to bacterial metabolism and is used and depleted thereby. The extent of this depletion was not determined, as no data on phosphates in the ground water before recharge were available.

Phenols were determined in relatively few samples. The reclaimed water contained an average of 0.001 ppm - 0.002 ppm. The data are insufficient and the experimental error too great at these low concentrations for any effect of underground travel on the phenol content to be determined with certainty. Certain strains of soil bacteria are capable of utilizing phenol in highly dilute solutions as a nutrient.

Detergent concentrations (as alkyl benzene sulfonate) were reduced from about 10 ppm in the Hyperion plant influent to 4 ppm in the HRAS effluent. Intermittent sand filtration further reduced them to about 1+ ppm.

The fate of detergents in ground water was of interest. Consequently, special efforts were made to collect numerous observation well samples. Additional analyses were made through special arrangements with the Los Angeles County Sanitation Districts in their laboratories. The data would appear to indicate that detergents are relatively stable underground. It may, therefore, be necessary to accomplish their reduction largely by means of aerobic tertiary processes to a final concentration of about 1 ppm.

No standards have been promulgated for detergents. Recent work by the United States Public Health Service (USPHS) indicates their undesirability because of taste and foam-producing properties, even in low concentrations. Flynn, et al, report[9] that frothing appears at about 1.0 ppm and that tastes are detectable by most persons at 1.5 ppm.

The biochemical oxygen demand of a water is a generally accepted measure of its decomposable organic content. Ground water containing a high BOD generally develops a stale, musty taste. Tertiary treatment reduced the BOD of the water prior to injection to only 1.2 ppm. No problems were encountered at this low concentration. Many surface water supplies are higher in BOD.

Bacterial Quality.—During the earlier well runs, chlorine was applied at dosages up to 40 ppm. As a result, free chlorine residuals of 10 ppm or more were obtained at the recharge well tap, and corresponding samples from HR-20 and HR-50 likewise contained available chlorine. During Well Run No. 5 no residual appeared in any of the pumped samples because of the lower residuals

9 "Study of Synthetic Detergents in Ground Water," by J. M. Flynn, et al., Journal, AWWA, Vol. 50, No. 12, December, 1959, p. 1551.

(0.1 ppm - 0.3 ppm) in the injected water. Despite this, coliform counts essentially complied with USPHS drinking water standards (see Table 8 footnote). Plate counts were fairly uniform at each station throughout the entire run, indicating a condition of equilibrium between bacterial growth and sloughing in the aquifer.

Microscopic Investigations.—Routine samplings were made and microscopic assays conducted at the Hyperion Treatment Plant laboratories. Data were classified as bacteria, algae, zoological organisms, spores, organic matter, and inorganic matter. The resulting information has been tabulated and summarized in the district's files and describes (1) the number of samples analyzed, (2) the average aerial size, in microns, of each type, and (3) the relative percentage of the particular item to the total sample count.

A summary of these data indicates that the suspended solids encountered in the reclaimed waters from both the Hyperion high-rate and standard-rate activated sludge effluents were derived predominantly from bacterial slime masses. Bacteria that thrive in saturated soil under these conditions are saprophytic and non-pathogenic. Occasional algae, zoological organisms and other microscopic matter were noted in the injected water samples.

The microscopic examination of samples from the storage reservoir, Basin No. 3, indicates that slime bacteria in the sewage treatment plant effluent were largely removed in the basin and that the predominant suspended solids of the storage reservoir outflow were spore organisms of algal and zoological origin as well as some fully developed algae and certain zoological organisms.

As noted previously, the percolated waters collected from the spreading basin underdrains contained relatively few algae. The major microscopic matter reported in the filtrate consisted of spore forms, amorphous organic matter, cellulose, sand and some iron compounds. The recharge well samples during periods of high chlorination ranging from 11 ppm - 40 ppm contained iron oxides which are believed to have been the primary cause of clogging and fouling of the well. In the final months of the Hyperion tests from April through December, 1957, no significant iron oxide was encountered in the recharge well water samples when the chlorine dosage had been lowered to 0.5 ppm - 3.5 ppm.

Water samples from the observation wells contained few algal and zoological organisms. The major microscopic constituents of the well waters were spore forms, some cellulose, and during certain limited test periods, black amorphous matter.

Special Investigations.—Various special studies were conducted during the Hyperion tests to obtain further information on such subjects as evaporation, effect of sand filter depth, feasibility of redeveloping the recharge well, the practicability of using pilot plant lysimeters for obtaining test information, public health hazards and nuisances, and general operation information. Some of the results of these studies may be summarized briefly as follows:

1. Evaporation pan data indicated that the evaporation rate of reclaimed water was identical to that of conventional potable water.
2. The effect of sand filter depth was determined by samples from the spreading basin collection pans. The test data indicated that most of the suspended solids, organic removal, and stabilization occurred near the basin surfaces. However, some additional polishing occurred with increased sand filter depths. This confirmed data previously obtained at Whittier and Azusa.

3. No significant public health hazards or nuisances such as odors, flies, and mosquitoes were encountered. The appearance of the reclaimed water was indicative of its basic quality and suitability for recharge well use. In general, a sparkling clear odor-free and generally foam-free water was successfully accepted by the test recharge well, whereas secondary treatment effluents, unimproved by further polishing, were unacceptable. The recalimed water, as monitored at the injection well, was found to be entirely satisfactory with respect to color, odors, tastes, and freedom from foaming.

Estimated Costs.—Unit costs for sewage reclamation for recharge in wells are dependent on a number of variables including, but not limited to the following: Quality of waste water, the required capacity, availability and cost of land, the distance from a suitable source of sewage effluent to the recharge site, topography, and storage requirements.

In general, a temporary shutdown of a recharge operation is not detrimental, provided the waste water can be bypassed. Consequently, the only storage needed is a quantity sufficient for process treatment and to smooth out minor fluctuations in flow rates.

Calculations have been made for reclamation facilities designed for a flow of 50 MGD, delivered to a distribution system located within three miles of the Hyperion plant.

Provision was made in the calculations for a possible charge of $6 per MG for Hyperion standard-rate activated sludge effluent supply. This covers the extra sewage treatment expense needed to insure a constant high standard of treatment prior to reclamation that might otherwise not be required, and to provide special service in the form of monitoring for deleterious substances.

Costs have been estimated for a nominal-rate reclamation system based on intermittent surface spreading (Table 9). Surface spreading costs are shown despite the subsequent lack of adequate area near the Hyperion Treatment Plant, because of the successful results obtained with this method of waste water reclamation and its consequent general applicability for situations where land is available.

The cost data are based on conditions at the specific sites considered by the Los Angeles County Flood Control District for its utilization. Thus, for surface spreading, basins would have been constructed within two miles from the Hyperion plant. The reclaimed water would then have been pumped out of the ground and back to the recharge well distribution header in the vicinity of El Segundo, Calif., approximately 4 miles from the spreading site.

The costs have been estimated for delivery of the reclaimed water to the recharge barrier system. In additon, net costs, which include only the reclamation processes have been calculated.

CONCLUSIONS

A comprehensive, 3-yr investigation was completed by the Los Angeles County Flood Control District to determine the feasibility of reclaiming variable quality sewage effluent from the city of Los Angeles Hyperion Treatment Plant for injection well recharge. Following is a summary of the major conclusions resulting from this investigation:

1. Reclamation of high-rate activated sludge type sewage effluent from the Hyperion Treatment Plant by means of normal practices of continuous surface

TABLE 9.—SURFACE SPREADING RECLAMATION COSTS - 50 MGD

A. Capital Costs		
Basin construction, dikes, access roads, and so forth 200 acres @ $2,000		$ 400,000
Pumping station, 2 miles of pipeline, and distribution headers for delivery to spreading basin		745,000
Pumping station and 4 miles of pipeline to recharge well system		1,240,000
Chlorination facilities		140,000
Engineering, legal, and contingencies (20%)		505,000
Land - 200 acres @ $20,000 per acre		4,000,000
	Total	$7,030,000
B. Annual Costs		
Amortization of construction costs over 25 yr[a] at 5% interest (CRF = 0.0710)		$ 179,000
Labor for round-the-clock operation, including maintenance and supervision		110,000
Repairs, equipment, supplies, transportation, and so forth		52,000
Chlorine		45,000
Utilities, including power for pumping to 75 ft head		100,000
Power to pump to additional 265 ft head for recharge		290,000
Charge for treated sewage @ $2 per acre-ft		110,000
Administration		30,000
Interest on land, engineering, legal, and contingencies	5%	225,000
	Total	$1,141,000
Operating cost per MG	$62.52	
Cost per acre-ft (AF)	$20.84	
Operating cost of reclaimed water, excluding distribution to recharge system (AF)	$13.79	

[a] An over-all 25-yr life has been assumed because of the possibility of technological obsolescence.

spreading on well-sorted dune sand was unsuccessful due to gradual sealing off of the surface by anaerobic slimes. Neither chlorination of the effluent before spreading nor increased water depth resulted in practical improvement of percolation rates.

2. Continuous spreading accompanied by raking of the test basin surfaces permitted operation at average percolation rates of 0.6 cfs per wa - 1.0 cfs per wa for several months but demanded excessive maintenance. Reclaimed water produced by the wet-raking process fouled the experimental injection well.

3. It was found feasible to reclaim high-rate activated sludge effluent containing suspended solids and BOD as high as 87 ppm and 59 ppm, respectively, by means of intermittent surface spreading in small basins at an average of

0.5 cfs per wa. The water produced averaged 7.2 ppm suspended solids and 3.5 ppm BOD.

4. Intermittent spreading of an experimental standard-rate activated sludge (SRAS) effluent of higher quality than the high-rate activated sludge (HRAS) plant effluent yielded a reclaimed water containing 2.5 ppm - 5.7 ppm suspended solids and 0.3 ppm - 1.2 ppm BOD.

5. Storage of Hyperion effluent in an oxidation basin for 2 days prior to surface spreading improved its quality but had no significant influence on the recharge characteristics of the final reclaimed water.

6. Water reclaimed by intermittent spreading was well-stabilized, of sparkling clarity and odor-free.

7. Water reclaimed by intermittent spreading in small basins of both HRAS and SRAS sewage effluent was successfully substituted for filtered potable water in a test recharge well and displayed similar hydraulic characteristics over a 6-month period.

8. Injection well tests demonstrated that a suitable injection water could contain about 6 ppm suspended solids and 2 ppm BOD, provided the solids were of a suitable particle size, fully oxidized, and contained a minimum of inert clay or silt.

9. Intermittent filtration in spreading basins followed by chlorination produced a reclaimed water that met the chemical and bacteriological standards for potable water promulgated by the USPHS, and the irrigation water standards of the United States Department of Agriculture.

10. No significant public health problems including odors, flies, or mosquitoes occurred during the Hyperion tests.

11. Chlorine dosages sufficient to maintain a free chlorine residual of 0.1 ppm at the injection well appeared suitable for effluent injection operation, while at the same time affording adequate protection against contamination of ground water.

12. Detergents were reduced by conventional waste treatment followed by intermittent sand filtration of an average of 1.12 ppm. No foaming nuisance was encountered at this detergent concentration.

13. Standard-rate activated sludge effluent containing 25 ppm suspended solids was entirely unsatisfactory for direct recharge into an injection well. Additional or tertiary treatment was found necessary.

14. The method originally planned by the Los Angeles County Flood Control District for waste water reclamation by surface spreading has been abandoned because the only potentially adequate spreading ground site in the vicinity of the Hyperion Treatment Plant was subsequently acquired by the Los Angeles City Airport Commission, and it was necessary, therefore, to seek feasible methods of rapid-rate reclamation.

15. Special investigations on a number of potential processes for rapid-rate reclamation were undertaken, including intermittent filtration through coarse gravel, rapid sand filtration, trickling filter operation, limited storage in an open reservoir, and up-flow sludge blanket, all accompanied by both shock and continuous chlorination. The technical operating problems encountered with these methods were not solved within the test period, although it appeared that rapid sand filtration at rates up to 200 cfs per wa would be feasible.

16. Water quality achieved by rapid sand filtration averaged 6.5 ppm suspended solids and 2.4 ppm BOD during one 7-week run. Further investigation of this and other high-rate processes is indicated.

17. Test observation wells located 20-ft, 50-ft, and 70-ft distance from the injection well enabled the collection of injected water samples that had traveled through the restricted aquifer. After correcting for ground water dilution, the nitrogen, phosphate, and ABS detergent concentrations in the reclaimed water were found to decrease measurably with lateral distance from the injection well.

18. Both prototype and lysimeter test facilities were useful in obtaining realistic waste water operation data applicable to full scale test procedures.

19. Preliminary cost estimates indicate that a 50 MGD reclamation plant can provide water at a cost of about $20.84 per acre-ft utilizing intermittent sand filtration spreading. This cost includes $2.00 per acre-ft charges for sewage effluent and charges for pumping to a head of 265 ft to supply the recharge well distribution system four miles from the spreading area. Capital cost outlays are calculated at $7,030,000, based on estimated land costs of $20,000 per acre. The net costs of reclaimed water excluding the final pumping and distribution system costs are estimated to be $13.79 per acre-ft for intermittent sand filtration.

During 1961, the Los Angeles County Flood Control District will initiate a new study at Hyperion, predicated largely on the conclusions presented herein. The scope of this study will be an evaluation of the rapid sand filter and other processes for waste water reclamation, and the degree of polishing necessary to obtain acceptance of the produced waters in a series of recharge wells.

ACKNOWLEDGMENTS

During the research work described herein, H. E. Hedger was Chief Engineer of the Los Angeles County Flood Control District until January, 1959. M. E. Salsbury succeeded Hedger in directing the District's activities. Development of the Hyperion test project and its operation were under the direction of the senior author. Field supervision of the project was carried on by H. A. van der Goot (deceased) and A. E. Bruington. Ralph Stone served as consultant on sanitary engineering matters, and Lawrence A. Meyerson analyzed the field data and prepared the project report for the Flood Control District prior to his employment by the Regional Water Pollution Control Board.

No small measure of the success of the project is due to the engineering staff of the Los Angeles City Hyperion Treatment Plant then supervised by Chief Engineer and Assistant Chief Engineer, G. A. Parkes and Robert D. Bargman, as well as William F. Garber, Director of Laboratories. Substantial laboratory work was also contributed by the Foundation and Testing Division of the Flood Control District under the direction of F. O. Fricker, and by the Los Angeles County Sanitation Districts' engineering staff under the direction of A. M. Rawn.

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COLIFORM ORGANISMS AS AN INDEX OF WATER SAFETY

Progress Report

Committee on Public Health Activities

SYNOPSIS

Sanitary Engineering programs related to design, construction, operation, maintenance, supervision and regulation of water, from a public health protection standpoint, are largely based on coliform organism standards. Periodically, new situations arise necessitating re-evaluation of the standards and re-examination of experience and research related to acceptance of the coliform organism as the basis for determining microbiological safety of water. At the suggestion of the Executive Committee of the Sanitary Engineering Division, the Public Health Activities Committee has prepared this report to bring to practicing sanitary engineers a summary of current opinions and a capsule view of the scientific investigations relating to the question of the validity of coliform standards.

CURRENT STATUS

The United States Public Health Service (USPHS), with the assistance of its advisory committee, has completed a preliminary draft of the 1961 United States Public Health Service Drinking Water Standards. After considering all of the available data those concerned unanimously agreed the coliform group

Note.—Discussion open until April 1, 1962. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SA 6, November, 1961.

should continue to be the indicator of the microbiological safety of water supplies.

VIRAL DISEASES

Public health authorities agree that infectious hepatitis outbreaks have been caused by contaminated water supplies. One outbreak which is frequently referred to was the 1955 to 1956 epidemic in Delhi, India, when between 7,000 and 29,000 cases of infectious hepatitis occurred in a population of 1,700,000 persons. Investigators reported "The fact that no undue incidence of typhoid or dysentery occurred indicated the water treatment processes were adequate to deal with bacterial pathogens but not adequate to remove or kill the virus of infectious hepatitis." Reportedly, the water leaving the Delhi plant at the time the virus was present met acceptable bacteriological standards. The question then arose: Does a test that shows absence of coliform organisms or conformance with a coliform standard adequately indicate the water supply does not contain viable infectious hepatitis viruses?

A well-known major advantage to the use of the coliform organisms as a basis for standards is the relative ease of determining whether they are present in water. Isolation of virus from water is difficult. While laboratory procedures are available for detecting a number of enteric viruses that may be found in water, infectious hepatitis is not one of these, and its detection still is based on tests with human volunteers.

VIRAL RESISTANCE TO TREATMENT

In 1945, at Philadelphia, Pa., members of Civilian Public Service Unit No. 140 served as experimental subjects in studies conducted by representatives of the University of Pennsylvania and the armed forces. These volunteers consumed water innoculated with known quantities of coliforms and infectious hepatitis virus. Some of this contaminated water was subjected to various water treatment processes. Hepatitis syndrome developed in one or more volunteers who consumed the water in each type of treatment except in those cases where adequate chlorination inactivated the virus. However, treatment other than chlorination reduced[1] the incidence of infection by about 40%.

The report on this experiment concluded with the following:

> "(a) Coagulation settling and filtration (diatomite filter) of contaminated water did not eliminate or inactivate infectious hepatitis virus . . .
>
> "(b) The application to such water (previously coagulated, settled and filtered) of sufficient chlorine to provide, after 30 minutes contact, total and free residual chlorine concentrations of 1.1 and 0.4 ppm, respectively, apparently was adequate to inactivate the hepatitis virus under the conditions of this experiment. However, the same 30 minute residual total chlorine concentration (1 part per million) in contaminated water that had not been pre-treated by coagulation, settling and filtration, did not inactivate the infectious hepatitis virus."

The report further said complete control of the hepatitis virus and probably other infectious agents in drinking water depends almost entirely on the dis-

[1] "Inactivation of Virus of Infectious Hepatitis in Drinking Water," by John R. Neefe, James B. Baty, and John G. Reinhold.

infectant, as the virus is not eliminated or inactivated by coagulation, settling and filtration. Chlorine dosages adequate to inactivate bacterial pathogens occurring in drinking water may not be adequate for the hepatitis virus, particularly if the water has a high content of unoxidized organic material.

Paul W. Kabler, and his colleagues from the Robert A. Taft Sanitary Engineering Center in their paper presented before the 1960 meeting of the Laboratory Section of the American Public Health Association said in part:

> "Data indicate that at temperatures of approximately 20°C and pH values no higher than 8.0 - 8.5, a free chlorine residual of 0.2 to 0.3 mg/l will probably destroy most of the tested viruses in 30 minutes. At temperatures below 10° to 15°C and pH values greater than 8.5 effective virus kills with free chlorine residuals with 0.2 to 0.3 mg/l are probably not obtainable unless long detention periods are used."

They further reported that in one experiment at 25°C and a pH of 7, a concentration of at least 9 mg/l was necessary for inactivation of polio virus with a contact period of 30 minutes, and of 6 mg/l with a 1 hour contact period. They also reported:

> "The relative resistance of infectious hepatitis virus is not clear because sufficient contact times were not tested. It is apparent, however, that the different viruses exhibit wide variation in their susceptibility to free chlorine."

The Appendix "Enteric Viruses in Water and Wastes," prepared by the Robert A. Taft Sanitary Engineering Center for this report, provides a much more thorough coverage of this entire subject.

EXPERIENCES WITH VIRUS IN WATER

Investigators working with purified Coxsackie viruses have reported that free chlorine residuals of 0.27 ppm to 0.32 ppm are necessary to destroy that virus. They concluded that marginal treatment of water is not sufficient to destroy viruses even though it may reduce the coliform density to closely conform to established standards. They indicate that the coliform index should be interpreted with discretion and regard for available data on survival of viruses.

In 1959 Clark and Chang wrote that proven water-borne outbreaks of enteric virus diseases have occurred only with the virus of infectious hepatitis. There are no reports of water-borne outbreaks of disease caused by Coxsackie, Adeno-virus, or Echo viruses. This may, however, be the result of the general lack of information on the clinical illnesses that many of the viruses produce. The literature contained seventeen reports of epidemiologically established water-borne infectious hepatitis outbreaks, six of which occurred in the United States.

An inquiry about viral disease transmission by drinking water was sent to the health departments of 50 states and 10 Canadian provinces by the Purification Division Task Group on Viruses in Water (of the American Water Works Association). Out of 46 replies, 7 health departments had determined that one or more infectious hepatitis outbreaks in their areas of jurisdiction were water-borne. An additional 6 health departments indicated knowledge of one or more viral disease epidemics in their states which could have been water-

borne. The other health departments either concluded that there had been no water-borne disease outbreaks in their states or that they lacked information. The health department reports indicated that where water was shown to be the virus carrier, the water did not meet USPHS bacteriological standards.[2]

If one accepts the data on coliform organisms reported by the New Delhi plant, it appears that under unusual circumstances the current coliform index may be unreliable in judging the adequacy of modern water treatment processes for destroying or removing the virus of infectious hepatitis and perhaps other enteric diseases.

USE OF OTHER BACTERIA IN PLACE OF COLIFORM AS TEST INDICATORS

Probably ever since the coliform group was accepted as the standard indicator of fecal contamination, efforts have been made to find or to convince authoritative groups of the efficacy of other test organisms. Fred A. Rosenberg, in his Ph.D. thesis at Rutgers University, New Brunswick, N. J. did a literature review on "Coliforms and Enterrococci as Indicators of Water Pollution." His summary is an important part of the appendix of this report. A brief synopsis is given here without attempting to carefully document each source. The main coliform group rival for recognition as the basis of water microbiological standards in the enterococci. Because enterococci occur only in feces of humans and warm-blooded animals, and in nature only as an element of fecal contamination, and because their isolation has been made considerably easier through development of new media, some authorities believe they are superior to coliform organisms as contamination indicators. Reports of various investigators vary as to the relative counts of coliform and enterrococci in human feces. It is generally agreed that both are present in sufficient numbers to be used as indicators of fecal contamination with equal justification. Reports on survival time of the two organisms vary, some indicating that one and some that the other survives longest in nature. Examination of contaminated well water indicated that although the relative numbers of coliform and streptococci varied, the number of samples in which each was more numerous was about equal.

Prescott, Winslow and McCrody stated:[3]

> "There is good evidence however that streptococci are less resistant than coliforms to the unfavorable conditions which exist in water of ordinary organic purity. Savage and Wood, in an experimental study of the viability of streptococci in water, found that they died out in parallel with coliforms although a trifle more rapidly."

F. A. Rosenberg and H. Heukelekian[4] concluded that:

> "There is still uncertainty as to whether coliforms per se or Escherichis coli be used as an index of water quality. The difficulties encountered in a true measurement of Escherichia colonies present, together with

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2 "The Physiological and Health Aspects of Water Quality," presented at the June 6, 1961 AWWA natl. meeting in Detroit, Mich.

3 "Water Bacteriology," by Samuel T. Prescott, Charles-Edward A. Winslow, and Mac Harvey McCrody, 1946, John Wiley and Sons, New York, p. 213.

4 "Coliforms and Enterrococci as Indicators of Household Well Water Pollution," by F. A. Rosenberg and H. Heukelekian, paper.

recent advances in media development, which have made it relatively easy to perform analyses for the enterococci, suggest use of the latter as indicator organisms of fecal contamination."

More Experience and Research Needed.—The experience of Chanute, Kans., in the use and re-use of treated sewage treatment plant effluent might be cited as an example of the efficiency of complete treatment in the removal of pathogenic bacteria and viruses. This was reported by Metzler et al.[5] For a five month period of drouth in 1957, Chanute used the town's sewage treatment plant effluent to augment their meager water supply. During this time laboratory tests for pathogens and viruses in the finished water showed negative results and no cases of water-borne bacterial or virus infections were reported.

The Committee, however, recognizes that it is desirable to encourage further research to determine whether noncoliform indicators of contamination may usefully supplement coliforms for certain water uses.[6] As indicated in the abstracts of papers presented at the Rudolphs Memorial Conference, the question is "Shall we find it advantageous to use different microbiological indicators of contamination for different water uses?" This is, of course, a consideration that will be clarified by further research and experience.

Certain other limitations in the use of coliforms have sometimes been considered. For example, Paul Kabler at the Rudolphs Memorial Conference noted that of the four recorded epidemics of amoebic dysentery in the United States associated with water supply all were due to defective distribution systems. In other words, the cysts were not in the water leaving the treatment plant.

Ed Laubusch at the same conference noted that the resistance of the spores of Endamoeba histolytica to chlorine was so great that the absence of coliforms was no guarantee that the cysts were also absent. As a matter of fact, he indicated that where chlorination was the sole method of water treatment, the protection against amoebic dysentery was nil. These observations suggest that absolute dependence on the absence of coliforms would not be desirable. The profession must be alert to sanitary defects in the system that would permit the entrance of contamination against which chlorination is only partially or perhaps completely ineffective.

More needs to be done to determine what hazard, if any, exists from viruses transmitted to underground water supplies by re-use of partially treated wastes. Before this question can be effectively answered, more basic research must be done to find a practical way of determining whether or not pathogenic viruses have been effectively removed or destroyed. To this end the Public Health Activities Committee recommends high priority for designing adequate research projects and devoting an appropriate amount of thought, energy and funds to finding answers to this important question.

CONCLUSIONS

1. In spite of questions and suggestions for a substitute test there is at present no justifiable basis for changing the bacteriological criteria for water

[5] "Emergency Use of Reclaimed Water for Potable Water Supply at Chanute, Kansas," by Dwight F. Metzler, et al., Journal, AWWA, August, 1958.

[6] Communications and Reports, Robert A. Taft Engrg. Center, Submitted by Bernard Berger.

supply safety and no need for serious concern with the safety of a water supply which fully meets USPHS Drinking Water Standards.

2. Experiences with infectious hepatitis outbreaks and laboratory data lend support to the already generally accepted principal there may be a serious health hazard created by accepting "marginal treatment" of water supplies, especially those subject to sewage contamination.

3. Because infectious hepatitis virus survive all normal steps in water treatment except proper free residual chlorination and since viruses survive heavy doses of combined chlorine, the practice of providing free chlorine residual treatment with adequate contact is important, especially when a water supply is known to have been subject to fecal contamination.

Respectfully submitted,

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Public Health Activities Com.

APPENDIX I

At the request of committee member Bernard B. Berger the committee includes as part of its report the following authoritative statement on Enteric Viruses in Water and Wastes, prepared at the Robert A. Taft Engineering Center of the United States Public Health Service.

OCCURRENCE AND SURVIVAL

During the past 15 yr more than 70 viruses have been recognized that may be present in human feces and, therefore, may occur in domestic sewage. Poliovirus has been reported in concentrations as high as 100,000 tissue culture units per gram of feces by Sabin.

Neefe and associates estimated there were 10,000 to 100,000 infectious doses of infectious hepatitis virus per gram of feces in human cases. Kelly and Sanderson found 2 to 44 plaque-forming units (PFU) of viruses per 100 ml of raw sewage and identified 15 strains of Coxsackie, ECHO and polioviruses. Highest densities were recorded in August and early September with greater concentrations in samples of raw sewage than in treated sewage. Demonstrations of poliovirus by Levaditi, Paul and Trask, and Bloom and associates, as well as by Kelly and Sanderson, and of ECHO and Coxsackie viruses by Melnick and co-workers, Metzler, et al., and Paffenbarger and associates have all been qualitative. Of 150 viruses isolated by Bloom, et al., 31 were identified as ECHO viruses, 4 as poliovirus, 76 as Coxsackie viruses,

17 were not identified and 22 were lost in culture. Enteroviruses were found in all stages of sewage treatment plant operation, including the final effluent.

Toomey, Takas and Weaver isolated a polio-like virus from creek water in Ohio by direct transmission to a cotton rat, and Kling demonstrated poliovirus in a well water in Sweden. An unclassified adenovirus was recovered from a water sample taken from an ejector-type drinking fountain at a boys camp in Paffenbarger's studies, and enteroviruses were isolated from Hudson River water by Kelly and Sanderson 400 ft from the Albany sewage plant outfall.

In laboratory studies by Clarke and associates, Coxsackie A2 virus survived 61 days in sewage at 8°C and 41 days when stored at 20°C. In distilled water at 8°C, the virus survived more than 272 days, and at 20°C, 41 days. In raw Ohio River water at 8°C, it survived 16 days, and at 20°C only 6 days; in autoclaved Ohio River water, the survival times were more than 171 days and 102 days, respectively. Rhodes, et al., demonstrated poliovirus in experimentally contaminated river water after 188 days storage. Gilcress and Kelly found that Coxsackie A5 survived 280 days when stored in water at 8°C to 10°C. The ability of contaminated well water to infect human volunteers with infectious hepatitis indicated that the virus was capable of at least 10 weeks survival in Neefe's studies. It appears that, in general, enteric virus survival is longer in treated or clean water than in polluted water.

REMOVAL OF VIRUSES BY SEWAGE TREATMENT PROCESSES

Carlson, et al., added mouse adapted poliovirus in the form of infected mouse brain and cord suspension to mixed activated sludge liquor in a ratio of one part virus to 300 parts of sludge and aerated the mixtures 0, 6 and 9 hours. Using sludge concentrations of 1,100 ppm, 2,200 ppm, and 3,300 ppm their results indicate that activated sludge in amounts as low as 1,100 ppm, with 6 hr retention, removed or inactivated the virus to a point at which infectivity for mice was greatly reduced. (See Table 1.)

Gilcreas and Kelly reported that the level of Coxsackie A virus was reduced about 60% in sewage treated by the trickling filter process. In a later publication, Kelly, et al., found that sedimentation in an Imhoff tank having a 2-hr retention time had no destructive action on the virus and that secondary treatment on a trickling filter followed by sedimentation and chlorination did not always effectively destroy virus in sewage. Mack, et al., noted a higher percent recovery of viruses (poliovirus, Coxsackie, ECHO) from settled activated sludge samples than from the liquid phase of sewage samples, indicating perhaps that activated sludge process may be effective in removing viruses from sewage.

Using a bench model activated sludge unit with 6 hr to 7 hr retention, Clark, et al., in 1960, removed approximately 90% Type 1 poliovirus and about 98% Coxsackie A9 virus. Treatment with 200 ppm sludge showed less virus removal than with more concentrated sludge. In primary treatment studies no significant loss of virus occurred within 3 hr settling time, but some loss was observed between 6 hr and 24 hr of settling. Also, in 1960, Kelly and Sanderson reported that enteroviruses (poliovirus, Coxsackie and ECHO in domestic sewage) were reduced by primary treatment 0 to 80%; by trickling filter 50% to 90%; and by chlorination 80% to 95%. Peak densities of the viruses occurred

in August and September, with minimal or undetectable numbers from December to May.

REMOVAL OF VIRUSES BY WATER TREATMENT PROCESSES

Flocculation.—The removal of viruses from water by chemical flocculation appears to be the result of a metal cation-protein reaction in which a metal-virus complex is formed and aggregates to form a precipitate. The reaction is, therefore, nonspecific and the results obtained with one animal virus should be applicable to other animal viruses. On the other hand, the degree of virus removal depends entirely on the efficiency of the flocculation process.

TABLE 1.—REMOVAL OF ENTERIC VIRUSES BY SEWAGE TREATMENT

Investigator	Date	Organism Studied	Percentage of Organisms Removed by				Remarks
			Trickling Filter	Activated Sludge	Anaerobic Digestion	Chlorination	
Carlson, Ridenour, and McKhann	1943	Mouse adapted poliovirus	-	Mostly removed	-	-	Aerated 6 hr-6 of 30 mice died
Gilcreas and Kelly	1954	Coxsackie A	60	-	-	-	
Kelly, Clark and Coleman	1955	Coxsackie	Reduced	-	Survive	-	More numerous in late summer
Mack, Mallmann, Bloom, and Krueger	1958	Poliovirus I and III	-	Survived	-	-	Virus isolated more often from sludge than from liquid
		Coxsackie	-				
		ECHO	-				
Clarke, Stevenson, Chang and Kabler	1960	Coxsackie A9	-	98	-	-	6 hr-7 hr retention
		Poliovirus I	-	90	-	-	6 hr-7 hr retention
Kelly and Sanderson	1960	Polioviruses) Coxsackie) ECHO	50-90	-	-	80-95	-

The pertinent information available on this subject is summarized in Table 2. It is apparent that there is wide variation in virus removal efficiency of the flocculation process as studied by several workers. It is of interest that the high virus removal attained by Chang and others was achieved with low dosages of aluminum sulfate, whereas low efficiency was attained by others who used high dosages. This clearly indicates that dosage of flocculant alone is not a measure of the efficiency of the process. A critical comparison of the virus removal efficiencies of flocculation processes cannot be made unless one knows the conditions under which the processes are used. As most

investigators have not indicated the relative performance of the flocculation process as judged by visual inspection of turbidity removal, the relatively low virus removals reported might suggest that the flocculation process was inadequate.

Results of a recent study by Chang, et al., indicate that if the flocculation process is satisfactorily applied, more than 95% removal of added virus should be obtained with 15 ppm-25 ppm of alum or ferric chloride in moderately polluted water having a turbidity of less than 260 ppm. In doublestage flocculation,

TABLE 2.—REDUCTION OF HUMAN ENTERIC VIRUSES IN WATER BY CHEMICAL FLOCCULATION

		Flocculation Process			
Virus	Type of Water	Stages	Coagulant Added[a] ppm	Floc Amount or Description	Virus Removal
Polio	Tap	1	100		Little[b]
	Raw	1	100		Some[c]
Polio	River	1	136		Some to significant[d]
		1	410		Some
		1	136-273	0.4-1.03 ml per l	Little to some
		1	273-546	1.5-2.2 ml per l	Significant
Infectious hepatitis[e]	Distilled	1	69		Little to some
Coxsackie A5	Spring	1	28-45		Some
Coxsackie A2 (purified)	Ohio River (16-255 ppm turbidity)	1	15	Good	95.7% at 25° C
		1	25	Very good	95.9% at 5° C
		2	15	Very good	98.6% at 25° C
			15*	Very good	99.8% at 25° C
		2	25	Very good	99.6% at 5° C
			25*	Very good	99.9% at 25° C

a Alum except where marked with an asterisk; asterisks indicate ferric chloride.
b Probably less than 25%.
c Probably less than 50%.
d More than 50%.
e Gauze-strained fecal suspension in distilled water.

with the same dosage of flocculants, more than 99% removal should be expected. The percentage removal may be slightly lower in the cold than in the warm months. These experiments also showed that virus removal roughly parallels the removal of coliform, total bacteria, and turbidity. It is worth noting that virus is not destroyed by the flocculant but merely concentrated in the floc.

Chlorination.—Because chlorine reacts with organic matter and forms combined chlorine compounds, reliable data on virus destruction depend on the chemical definition of the test water and determination of the types of titrable chlorine present, which are markedly influenced by the pH of the substrate. The rate of kill under fixed test conditions is proportional to the concentration of the kind of chlorine available. Sufficient demand-free virus material should be present to allow for accurate determination of the inactivation rate. Because many of the earlier studies on the destruction of viruses by chlorine did not meet these criteria, the resulting data are not included in this review. The reports before 1946 did not differentiate between free and combined chlorine, and many of the reports also showed such a high chlorine demand of the test system that the data on the viricidal efficiency of chlorine cannot be accurately interpreted.

Free Chlorine.—Information on the viricidal efficiency of free chlorine in water is presented in Table 3. The amounts of free chlorine recorded in this table for the studies of Chang, Neefe and Lensen refer to the initial concentration. In each test the residual chlorine values were reduced to less than 1 mg per l. In the other investigations, the residual chlorine was not substantially less than the initial content.

The data of Weidenkopf are probably most accurate because the test conditions are well defined and the plague count technique provides an accurate estimation of virus concentration. Taking into consideration differences in procedures of the several investigators, the information in Table 3 reveals several significant points.

It is indicated that Theiler's virus is more resistant to the action of free chlorine than Coxsackie A2 virus, which is more resistant than poliovirus Type 1. Adenovirus Type 3 shows about the same resistance as do E. coli cells. The relative resistance of infectious hepatitis virus is not clear because sufficient contact times were not tested. It is apparent, however, that the different viruses exhibit wide variations in their susceptibility to free chlorine. The pH exerts marked effects on the viricidal efficiency of free chlorine. Weidenkopf's results show that lowering the pH from 7.0 to 6.0 reduces the required inactivation time by about 50%, and the findings of both Weidenkopf and Clarke indicate that a rise in pH from 7.0 to 8.3-8.9 increases the inactivation period about six times.

In the destruction of viruses by chlorine, Clarke's work suggests that the temperature coefficient for a 10° change (Q_{10}) is in the range of 2 to 3, indicating that the inactivation time must be increased 2 to 3 times when the temperature is lowered 10°C. Data in Table 3 also indicate that the chlorine concentration coefficient (N) lies in the range of 0.7 to 0.9. This means that the inactivation time is reduced a little less than half when the free chlorine concentration is doubled. To increase virus kill, therefore, there is some advantage in increasing the contact time instead of raising the chlorine content.

Kelly and Sanderson also observed that the viricidal efficiency of free chlorine is dependent on pH, temperature, chlorine concentration and exposure time. In addition, they pointed out that complete inactivation of some enteric viruses cannot be attained by exposure to 0.2 mg per l chlorine for 10 min at pH 7.0 (usual bacterial parameters of disinfection) at an initial level of 300 to 3000 virus doses per ml in the water.

TABLE 3.—VIRICIDAL EFFICIENCY OF FREE CHLORINE IN WATER

Investigator	Virus	Temp. °C	Final pH	Free chlorine mg per l	Virus destruction
Chang et al.	Part. purif. Theiler's virus in tap water	25-27	6.5-7.0	4.0-6.0	98.6% in 10 min
		25-27	6.5-7.0	4.0-6.0	99% in 5 min
Neefe et al.	Feces-borne Inf. Hepat. virus in dist. water	Room	6.7-6.8	3.25	30 min cont. time protected all of 12 volunteers
Lensen et al.	Purif. Polio II in dist. and lake water	19-25	7.4-7.9	1.0-1.5	10 min cont. time protected all of 164 inoc. mice
Clarke and Kabler	Purif. Coxsackie A2 in demand-free water	3-6	6.9-7.1	0.58-0.62	99.6% in 10 min
		3-6	6.8-7.1	1.9-2.2	99.6% in 4 min
		3-6	6.9-7.1	3.8-4.2	99.6% in 2 1/2 min
		3-6	8.8-9.0	1.9-2.0	99.6% in 24 min
		3-6	8.8-9.0	3.7-4.3	99.6% in 9 min
		3-6	8.8-9.0	7.4-8.3	99.6% in 5 min
		27-29	6.9-7.1	0.16-0.18	99.6% in 4 min
		27-29	6.9-7.1	0.44-0.58	99.6% in 3 min
		27-29	8.8-9.0	0.10-0.18	99.6% in 10 min
		27-29	8.8-9.0	0.27-0.32	99.6% in 7 min
		27-29	8.8-9.0	0.92-1.0	99.6% in 3 min
Weidenkopf	Purif. Polio I (Mahoney) in demand-free water	0	6.0	0.39	99.6% in 3 1/2 min
		0	6.0	0.80	99.6% in 1 1/2 min
		0	7.0	0.23	99.6% in 8 min
		0	7.0	0.53	99.6% in 4 1/2 min
		0	8.5	0.53	99.6% in 16 min
		0	8.5	1.95	99.6% in 7 1/2 min
		0	8.5	5.00	99.6% in 3 min
Kelly and Sanderson	Purif. Polio I (Mahoney) in demand-free water	25-28	7.0	0.21-0.30	99.9% in 3 min
		25-28	9.0	0.21-0.30	99.9% in 8 min
	Purif. Polio III (Saukett) in demand-free water	25-28	7.0	0.11-0.20	99.9% in 2 min
		25-28	9.0	0.11-0.20	99.9% in 16 min
	Purif. Coxsackie B5 in demand-free water	25-28	7.0	0.21-0.30	99.9% in 1 min
		25-28	9.0	0.21-0.30	99.9% in 8 min
		1-5	7.0	0.21-0.30	99.9% in 16 min
		1-5	8.0	0.21-0.30	99.9% in 30 min
Clarke et al.	Purif. Adenovirus 3 in demand-free water	25	8.8-9.0	0.20	99.8% in 40sec-50sec
		25	6.9-7.1	0.20	99.8% in 8sec-16sec
		4	8.8-9.0	0.20	99.8% in 80sec-100sec
		4	6.9-7.1	0.20	99.8% in 8sec-10sec

In low chlorine residuals, the virus inactivation rate is markedly affected by temperature and pH. Data indicate that at temperature of approximately 20°C and pH values no higher than 8.0-8.5, a free chlorine residual of 0.2 mg per l to 0.3 mg per l will probably destroy most of the tested viruses in 30 min. At temperatures below 10°C to 15°C and pH values greater than 8.5 effective virus kills with free chlorine residuals of 0.2 mg per l to 0.3 mg per l are probably not attainable unless long detention periods are used.

Combined Chlorine.—Trask, et al., found that 5.4 mg per l and 8.1 mg per l of residual chlorine as chlorine-ammonia were required to destroy 90% of Theiler's virus (FA strain) in 30 min, and in 10 min when the chlorine-ammonia weight ratio was 1 to 4. The temperature and pH of the treated water were not stated. At 26°C, a pH of 6.8 to 7.1, and a chlorine-ammonia weight ratio of 2 to 3, Chang, et al., observed that 5.2 mg per l of residual chlorine killed only 85.7% of Theiler's virus in 60 min. Trask and his associates also stated that 12.1 mg per l of titrable chlorine as azochloramid destroyed 90% of Theiler's virus in 30 min, and Fair, et al., found that 18 mg per l to 20 mg per l titrable chlorine as monochloramine T killed 60% to 80% of Theiler's virus in 60 min at 27°C and pH of 7.0 to 7.2. These data clearly indicate the low viricidal efficiency of these compounds.

Kelly and Sanderson showed that polioviruses (Type 1, MF 500 and Mahoney strains) and Coxsackie virus (Group B, type 5, EA 80) in water were inactivated by combined residual chlorine, the effective concentration depending upon pH, contact time and strain of virus. In general, longer contact was required to destroy the Type 1 poliovirus than the Coxsackie B virus in this study, and increasing the pH decreased the rate of inactivation. At 25°C and a pH of 7, a concentration of at least 9 mg per l was necessary for inactivation of poliovirus with a contact period of 30 min, and of 6 mg per l with a 1-hr contact period. Five-tenths mg per l required a contact period of more than 7 hr. These data make it clear that present methods of disinfecting water with combined chlorine may not be adequate to destroy the more resistant virus strains.

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APPENDIX II.—LITERATURE REVIEW

In substitution for a "reading bibliography" the committee wishes to include the following summary report of Fred A. Rosenberg, submitted by committee member Maurice A. Shapiro.

COLIFORMS AND ENTEROCOCCI AS INDICATORS OF WATER POLLUTION-SURVEY OF THE LITERATURE

Coliform Test.—The continued use of the coliform organisms as an indicator of fecal pollution of water has been, and is, the subject of widespread controversy. Gilcreas and Kelly maintained that it was still a reliable indicator of significant pollution. Norton felt that a water supply showing any bacteriological evidence of surface contamination or past pollution should not be approved for public use. Other authors were dubious about using the coliform test as an indicator of pollution, especially for rural supplies. Mallmann considered the test satisfactory for measuring sanitary quality of municipal water supplies, as not necessarily indicating unsafe conditions. France believed that a dependence on the coliform test along resulted in too many samples being condemned, and Fuller supported this argument with experimental evidence.

The view has been advanced that the vague indications given by determinations of the coliform group as a whole cannot be considered fully satisfactory, and that sanitary quality of waters is dependent on the numbers of E. coli present, rather than on the total number of coliform organisms. Perry and Bayliss have shown the apparent superiority of E. coli as an indicator of significant fecal pollution. Stuart believed that bacteriological examinations per se do not provide an absolute answer to suitability of water for human consumption, but must be supplemented by sanitary survey.

Perhaps one of the reasons why the coliform test has maintained its popularity, despite obvious shortcomings, was made clear in a paper by Butterfield. He noted that bacteriologists have tried to use a variety of organisms as indicators of fecal pollution, among them the various enteric pathogens, fecal streptococci, various anaerobes found in the intestinal tract of man, and a host of others. In no instance, however, has any of them been adopted as a sole indicator of water quality. This is due, the author believed, to the difficult procedures required in identification and enumeration, the inability to differentiate between fecal and non-fecal types, and the fact that some of the organisms, such as Pseudomonas aeruginosa, are not always encountered in human feces.

Because enterococci occur only in the feces of humans and warm blooded animals, and in nature only as an element of fecal contamination, and because their isolation has been made considerably easier through development of new media, they may prove to occupy the position of sanitary significance now held by the coliforms.

Presence of Enterococci, Escherichia Coli and Aerobacter Aerogenes in Feces.—Houston and Clemesha stated that coliforms and streptococci are present in feces in about equal numbers, the former being slightly more numerous. Winter and Sandholzer, using their own medium, found an average enterococcus count of 140,000 per gram for nine samples of human feces. This was approximately 1/100th of the coliform count. Ostrolenk and Hunter found enterococci to be more numerous in 37% of fecal samples, and E. coli to predominate in 63%. Guthof found the average enterococcus count in feces to be about the same as E. coli, and even slightly higher in some cases. He maintained that the different ratios of enterococci to coliforms, which were obtained by various authors, were due to the different media used in their analysis. Sullivan found enterococci in human feces to range from 9,000 to 18,000,000 per gram, with an average of 4.5 million organisms. Enterococci outnumbered coliforms in seven out of nine samples, taken from brooks, rivers, wells, and a reservoir. Aerobacter aerogenes made up 16% of the organisms in sewage samples and 16% of those in water, but the organism was not isolated from any fecal samples. Reedy and Punchocar, on the other hand, isolated Aerobacter in 87% of 253 fecal samples, and Burke-Gaffney in 8% of 500 fecal samples.

Results suggested that Aerobacter and intermediates composed the normal flora of non-fecal sources, whereas Escherichia was normally an inhabitant of feces. The ascendancy of Aerobacter and intermediates in feces stored at low temperatures seemed to be an expression of the lower optimum temperature for these groups, again suggesting their non-fecal origin.

Because enterococci do not proliferate in nature, as do some coliforms, Lattanzi and Mood felt that this fact made their presence a good indicator of recent pollution.

The environmental origin of coliforms of different types was considered of little significance as it had been shown that the so-called non-fecal types, such as Aerobacter aerogenes and A. cloacae are almost always present in small numbers in feces. Their presence in soil and water, in the absence of recent fecal pollution, may be established, but Bardsley found that they were not widely distributed in nature except where fecal pollution had occurred at one time or another. Clemesha showed that A. aerogenes is present in feces in small numbers, but that this organism, though rare at first, is extremely common after a few days. Rogers, Winslow and Cohen and Platt showed that A. aerogenes was able to survive longer than E. coli in running water and polluted streams. Parr also found E. coli most predominant in fresh feces, but storage at 37°C for 2 months resulted in replacement by aerogenes types.

Though Tonney and Noble showed, by means of a sanitary survey, that occurrence of A. aerogenes in the absence of E. coli is not necessarily associated with fecal pollution, Maassen, et al., believed that since aerogenes can be isolated from feces and urine, its presence in water need not be completely innocuous. This point of view was reiterated by Silberstein, et al., as a result of their investigations in Israel. Prescott, et al., stated that a large proportion of Aerobacter organisms usually accompany any Escherichia coli present in unsafe waters, and Gray felt the predominance of A. aerogenes over E. coli indicative of soil contamination or past pollution. This view was adhered to by Bardsley and Parr, the latter stating that when both organisms coexist in an environment, aerogenes survives longest.

Tonney and Noble, studying the relative survival rate of E. coli and A. aerogenes from feces and cultures under winter conditions, showed the E. coli count to have a more consistent relation to the initial amount of fecal pollution than A. aerogenes. The latter tended to increase more rapidly in nature and this may account for its predominance. The authors felt, that under such circumstances, the A. aerogenes count, if accepted as a quantitative index of fecal pollution, would tend to distort the sanitary picture, yielding no evidence as to how recently contamination had occurred. Brown and Skinner considered A. aerogenes as having no value as an index of fecal pollution. Platt confirmed that A. aerogenes is more resistant than E. coli except at 0-2°C, and Winslow and Cohen showed mixtures of E. coli and A. aerogenes organisms in water to give a 98% to 99% reduction in 10 days; E. coli death rate being higher after the initial 24 hr.

Intermediates may be nearly as frequent in polluted water as E. coli and may exceed the latter when pollution is only slight. Par maintained that intermediates occur in stool specimens to such an extent that they must be considered for sanitary purposes as indicators of pollution, and further noted that pollution may reach water in the absence of recoverable E. coli or the coli-intermediate-aerogenes group.

The lack of a basis of agreement as to which organisms may be considered essentially fecal in origin prompted Kichenko to forward the opinion that all varieties of coliform bacteria can serve as indicators of fecal contamination of water, and to list certain properties which he considered distinctive of fecal origin.

Citrate Utilization by Escherichia Species.—Koser's work noted that typical E. coli of fecal origin were unable to utilize sodium or potassium citrate when supplied as a sole source of carbon. Aerogenes-cloacae strains, on the other hand, which represented the largest section of cultures obtained from soil, all utilized citrate readily. Pawan found citrate negative strains in 98% of cow and human feces and citrate positive strains in 100% of unpolluted water. Another paper by Koser noted that the citrate utilization test showed some degree of correlation with sanitary surveys of water supplies. Citrate negative cultures constituted 64.5% of coliforms from polluted water and 16.7% of those from waters of high sanitary quality.

Parr held that when fecal specimens contain citrate utilizers as well as E. coli, the citrate utilizers multiply in storage much more rapidly than do the coli, eventually replacing them. Fischer found E. freundii restricted by the growth of E. coli.

E. Coli Bacteriophage as Pollution Indicator.—In experiments with bacteriophage, E. coli Brx was used as an indicator organism. The phage content in well waters was found to be much higher in summer than in winter, and increased after periods of rain. Wells which, as a result of poor construction, were more liable to contamination by surface water showed greater numbers of phage.

Persistence of E. Coli and Enterococci.—Mallmann and Litsky showed that enterococci disappeared after 33 days from sewage treated soil, coliforms persisting throughout the 11 week study. Ostrolenk, et al., on the other hand, noted that enterococci in artificially contaminated soils and normal feces appeared to survive longer than E. coli under identical conditions.

Seasonal Variation in Bacterial Numbers.—Bardsley, during 6 hr of water sampling, noted a decided tendency for the numbers of E. coli to remain con-

stant or decrease in winter and spring, whereas the percentage of Aerobacter and intermediates increased. In late summer or autumn, the reverse was found to be true. Thomas, et al., found the results for a series of samples from 11 water supplies showing a marked seasonal variation; E. coli being more numerous in summer.

Winslow and Phelps examined water samples from springs, wells, brooks and pools and found that whereas 3 of 86 gave more than 100 bacteria per ml in winter and spring, 16 of 58 samples taken in the summer gave more than 100 bacteria per ml. Prescott examined 147 shallow farmyard wells, 124 of which contained no E. coli and averaged 190 bacteria per ml. The 23 wells giving positive tests averaged 570 bacteria per ml. Investigation has shown that in midsummer bacteria capable of growing at 37°C are more abundant in normal waters than in winter and spring.

The increase in number of organisms present in wells after rainfall has been mentioned by Burke-Gaffney and Rudolfs.

Antagonistic Effects.—A large number of authors have discussed the antagonistic effects which must be considered in determining the number of coliforms present. Levine and Tanimoto found 19% of Escherichia strains isolated from feces and water to exhibit antagonistic properties against other gram-negative bacteria. Fredericq and Levine noted that Escherichia strains are only occasionally active against Aerobacter and Proteus.

Hutchison, et al., isolated organisms which were antagonistic to E. coli from 17% of water samples, and Etinger-Tulczynska likewise stated that large numbers of other members of the coli-aerogenes group resulted in the retardation of E. coli.

Relationship Between Coliforms and Enterococci.—In an examination of rural supplies for organisms of the coliform group, Bardsley examined a total of 1375 strains, of which E. coli comprised 762, A. aerogenes 256, and intermediates 257.

Morris and Weaver, using the Mallmann-Seligmann MPN method (Bacto-azide dextrose broth) in evaluating 52 wells, found 22 to contain more streptococci, 23 with a higher number of coliforms, and one with an equal number.

Leininger and McCleskey associated high total counts with high counts of coliforms, E. coli and enterococci. Using Litsky's azide dextrose broth, they found a mean ratio of 7.6 enterococci to 1 E. coli in river samples. Savage and Read tested deep and surface water supplies. In 67.7% and 65.7%, respectively, the absence of streptococci was associated with either the absence of E. coli or its presence in only very small numbers. However, the authors felt that because the absence of streptococci is less significant than their presence, it cannot be accepted to the same extent as the absence of E. coli as a reliable indication of freedom from serious contamination. The presence of streptococci, on the other hand, may be accepted as indicating considerable and recent contamination.

The greater resistance of enterococci to sodium salts has been noted by Chapman. Allen, et al., showed the death rate for E. coli and S. fecalis greatest at a pH of 6.1-7.1, and further concluded that the latter requires a much greater amount of nutrient broth than E. coli for initial growth.

Lattanzi and Mood reported a ratio of E. coli to enterococci of 63:1 using the Winter-Sandholzer technique with water taken from Long Island Sound at New Haven. Litsky, Rosenbaum and France found a ratio of 13.3 coliforms per enterococcus using ethyl violet azide broth.

Slanetz found the following mean ratios of enterococci to coliforms:

water	1.9:1
sewage	1:1.7
human feces	1:1.6
animal feces	15:1

Of 6 wells tested by the membrane filter and MPN methods, the following results were obtained:

Membrane filter:

enterococci higher than coliforms in 4 cases
coliforms higher than enterococci in 1 case
equal number in 1 case
ratio enterococci coliforms 1.6:1

MPN:

coliforms higher than enterococci in 3 cases
enterococci higher than coliforms in 3 cases
ratio enterococci coliforms 1:1.2

Croft reported on 185 water samples examined in different laboratories using azide dextrose-ethyl violet azide broth, buffered azide glucose glycerol-ethyl violet azide broth and M-enterococcus agar. M-coliform agar and the MPN technique were used for coliform enumeration. He found median ratios of enterococci to coliform of 1:5 in flowing water, 1:4 in impounded water, 1:2.5 in wells, and 1:10 in sewage.

Graziadei-Celoria found 5.91% of over 18,000 water samples to contain enterococci as compared to 14.74% with coliforms. In 92 samples, enterococci were present alone. Leiguards, et al., examined 1,000 water samples and found 332 to contain coliforms or streptococci. 83.1% of the indicator-organism containing samples contained coliforms; streptococci were present in 45.8%. Streptococci alone were found in 56 instances.

APPENDIX III.—CONCLUSIONS OF SPEAKERS PRESENTING PAPERS ON COLIFORM ORGANISMS AND ENTEROCOCCI AT THE RUDOLFS MEMORIAL CONFERENCE

(Prepared by B. B. Berger)

Three experts in the field presented papers on the subject: Joe McCarthy of the Lawrence Experiment Station, L. W. Slanetz of the university of New Hampshire, and W. L. Mallman of the University of Michigan; very briefly their conclusions were as follows:

McCarthy.—Coliform organisms are not universally effective as indicator organisms for the quality of drinking water. Two examples were cited: First, the Delhi outbreak in which the treated drinking water purportedly contained the virus of infectious hepatitis but was free of coliforms; second, the outbreaks of gastrointestinal illness in several locations where the microbiology

of the treated water did not show the presence of coliform organisms but instead showed large numbers of organisms not characterized by sheen production. McCarthy concluded, however, that the coliform test has stood the test of time and is the best indicator procedure we have, and we should stick to it.

Mallman.—Mallman described in detail the efforts over the last several generations to improve cultural procedures for the growth of enterococci. These efforts have been crowned with appreciable success. At the present time (1961) good methods are available for the production and enumeration of enterococci. Mallman urged that we go slow in supplanting the coliform index with one based on enterococci. He felt that for drinking waters derived from sources receiving sewage the coliform organism would probably be proved to be the best indicator organism. However, he did believe that for certain other waters cocci would have advantages. These waters included swimming pool waters, shallow wells, lakes characterized by considerable biological activity, and generally clean waters. Mallman particularly favored the use of cocci as the indicator organism for swimming pools, because here the major contamination was not enteric in nature but rather that due to throat and nasal washings. Mallman preferred the term "cocci" to "enterococci" because it is the more general term and includes the streptococci not normally flourishing in the intestine.

Slanetz.—Slanetz was very optimistic about the possibilities of using enterococci as indicator organisms. He stated that cultural procedures satisfactory for routine lab use are available, that studies had indicated the concentration of enterococci in feces to be of the same order of magnitude as that of coliforms, and that the membrane filter technique had been found to be a satisfactory procedure for capturing and enumerating the organisms. Slanetz emphasized two further advantages: (1) The presence of enterococci in water is direct evidence of mammalian fecal contamination and (2) there is good promise that current research will produce knowledge that will enable us to differentiate contamination from human sources from that by other mammals. This would be based on the types of enterococci strains present and their relative proportions. Slanetz concluded that, in his opinion, enterococci would supplant coliform organisms as the indicator organism of choice.

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EFFECT OF WATER TEMPERATURE ON STREAM REAERATION

Thirty-first Progress Report
Committee on Sanitary Engineering Research

SYNOPSIS

A search of the literature indicated that the effect of water temperature on stream reaeration rates had not been definitively established. Extensive studies of actual stream reaeration rates required knowledge of this effect. Consequently, it was necessary to undertake experimental evaluation of the temperature coefficient, (θ), of the reaeration coefficient, (k_2). This was done in a series of carefully controlled, high-precision, laboratory experiments that indicate the reaeration rate increases geometrically at the rate of 2.41% per °C.

INTRODUCTION

It has long been known from experimental evidence that deoxygenated water will absorb gaseous oxygen from the atmosphere at a higher rate if the temperature of the water is raised, other conditions being held constant. In the normal range of stream temperatures, a rise in water temperature results in a decrease in the viscosity, density, and surface tension and, conversely, the velocity of molecular vibrations is increased in the air and water at the interface. It is difficult to distinguish the exact role played by each of these factors, but it appears probable that the increased velocity of molecular vibrations is the most important. When undersaturated with dissolved oxygen, the

Note.—Discussion open until April 1, 1962. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SA 6, November, 1961.

film of water at the surface is saturated faster, molecular diffusion allows oxygen penetration to a greater depth (measured in molecular diameters) in equal time, at higher temperatures.

Because it is only the net over-all effect of temperature on reaeration that is of interest in most investigations, experiments have been conducted in the past by simply measuring reaeration rates at several temperature levels. Results of such experiments have been generally reported in the following form:

$$k_{2(T^\circ)} = k_{2(20^\circ)}\ \theta^{(T-20)} \quad \ldots\ldots\ldots \quad (1)$$

in which $k_{2(T^\circ)}$ is the reaeration coefficient determined under test conditions, T is the temperature in °C, $k_{2(20^\circ)}$ is the reaeration coefficient at the selected standard temperature (20°C), and θ is the temperature coefficient. The exponential form of Eq. 1 indicates a geometric (constantly proportional) increase in the reaeration rate as the temperature rises.

REVIEW OF PRIOR WORK

W. M. Black and E. B. Phelps[1] in 1911, noted an effect of temperature on the process of molecular diffusion, and further experimentation along this line in 1914 and 1916 was reported by Phelps.[2] W. E. Adney and H. G. Becker[3,4] and Becker,[5,6] in the pioneer work (1914 to 1924) leading to basic reaeration concepts, produced a substantial body of laboratory data on the effect of temperature. H. W. Streeter[7] used these data to develop his original formulation for the thermal adjustment of k_2 in the following form:

$$K_{2(T^\circ C)} = K_{2(20^\circ C)}\ \left[1.0159^{(T-20)}\right] \quad \ldots\ldots\ldots \quad (2)$$

which indicates the reaeration coefficient increases geometrically with temperature at the rate of 1.59% per °C. Streeter emphasized the preliminary nature of this value:

> ". . . for the present at least, as affording a reasonably accurate index of the influence of temperature variations on the rate of reaeration of streams."

and he goes on further:

> "This equation is proposed tentatively as probably representing most nearly, from available data, the effect of temperature variations on the value of the reaeration coefficient, K_2, under natural stream conditions."

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1 "Report on Discharge of Sewage Into New York Harbor," by W. M. Black and E. B. Phelps, Bd. of Estimate and Apportionment, New York, 1911.
2 "Stream Sanitation," by E. B. Phelps, John Wiley and Sons, Inc., New York, 1944.
3 Philosophical Magazine, W. E. Adeney and H. G. Becker, Vol. 38, 1919, p. 317.
4 Philosophical Magazine, W. E. Adeney and H. G. Becker, Vol. 39, 1920, p. 385.
5 Philosophical Magazine, H. G. Becker, Vol. 45, 1923, p. 518.
6 Industrial and Engineering Chemistry, H. G. Becker, Vol. 16, 1924, p. 1220.
7 Public Health Reports, H. W. Streeter, Vol. 41, 1926, p. 247.

Since that time this formulation has been widely used. H. E. Babbitt[8] refers to the original formulation given by Streeter. F. W. Kittrell and O. W. Kochtitzky[9] also used $\theta = 1.0159$ to correct observed values of k_2 to standard conditions. H. A. Thomas[10] used a similar base to construct a temperature correction table for k_2.

After his original analysis, however, Streeter changed his mind about this formulation, as is evidenced by the following passage from a publication by Streeter, H. W. Wright, and C. T. Kehr:[11]

> "Although this value (1.0159) was originally suggested as a correction factor for temperature variations in the specific rate k_2, a later review of Haslam, Hershey, and Keen's[12] data has indicated that it is more correctly applicable to variations in the rate of reaeration per unit of surface area; hence is directly comparable with the above values based on rate of (r_o)."

The rate (r_o) was defined by these authors as follows:

> "The rate (r_t) represents the observed amount of oxygen absorbed, in grams per square meter of water surface per day, as figured from the measured gain in D.O. content from Sta. 1-5 to each point. The rate (r_o) has been obtained by dividing the observed rate (r_t) by the average oxygen saturation deficiency in each stretch of the channel. It represents the maximum or "initial" rate of absorption at zero oxygen saturation, assuming that the rate is directly proportional to the percentage of saturation deficiency."

These researchers then undertook to apply data collected in two extensive series of artificial-channel reaeration experiments to the determination of a value for θ. In the first series of experiments reported in the paper by Streeter, Wright, and Kehr (velocities 1 to 4 fpm) the temperature coefficient was found to be 1.047 when referred to k_2, and 1.016 when referred to r_o. An examination of the data of these experiments shows little basis for their use in the determination of a temperature coefficient. No temperature control was used and, thus, the experimental conditions varied throughout the diurnal cycle. Data were grouped in wide temperature bands with, for instance, results of all tests on water between 10°C and 20°C being plotted at 15°C.

A second set of experiments reported in the same paper[11] was completed at higher velocities (10 to 40 fpm) and gave two sets of data that the authors considered amenable to the determination of the temperature coefficient, θ. These computations, reported only in terms of the initial rate, r_o, gave values of 1.0163 and 1.0167. The authors concluded from these computations:

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8 "Sewerage and Sewage Treatment," by H. E. Babbitt, John Wiley and Sons, Inc., New York, 1932.

9 Sewage Works Journal, F. W. Kittrell and O. W. Kochtitzky, Jr., Vol. 19, 1947, p. 1032.

10 Water and Sewage Works, H. A. Thomas, Jr., Vol. 95, 1948, p. 409.

11 Sewage Works Journal, H. W. Streeter, C. T. Wright, and R. W. Kehr, Vol. 8, 1936. p. 282.

12 Industrial and Engineering Chemistry, R. I. Haslam, R. L. Hershey, and R. H. Keen, Vol. 16, 1924, p. 1224.

"The close agreement between these two values and that obtained previously from the observations made on deaerated water in the continuous-flow channel suggests that a rounded value of (θ) approximating 1.016 fairly represents in a single figure the effect of temperature variations on the rate of stream flow reaeration, when expressed in terms of the amount of oxygen absorbed per unit of surface area or volume at a given constant degree of oxygen saturation.

This last statement implies that the temperature coefficient $\theta = 1.016$ can be applied either to r_o (a function of surface area) or k_2 (a function of volume). That this has been confusing can be seen in the way the reference has been used by other workers. Phelps[2] used the value from the lower velocity experiments in the formulation

$$k_{2(T°)} = k_{2(20°)} \left[1.047^{(T-20)}\right] \quad \ldots\ldots\ldots \quad (3)$$

whereas, on the other hand, O. J. O'Connor and W. E. Dobbins[13] cited Streeter, Wright, and Kehr[11] as reference for the equation

$$k_{2(T°)} = k_{2(20°)} \left[1.016^{(T-20)}\right] \quad \ldots\ldots\ldots \quad (4)$$

and went on to develop a theoretical formulation based on the Stokes-Einstein equation that gave results that varied only 1% to 3% from those found when $\theta = 1.016$.

The only other significant experimental work to be published in this field appears to be the much more recent papers of A. L. Downing and G. A. Truesdale,[14] and Truesdale and K. G. Vandyke.[15] Downing and Truesdale stirred water in beakers at various velocities and obtained the following results:

Stirring Rate (rpm)	Temperature Coefficient, in % per °C
12.8	2.02
38	2.38
86	1.73
115	2.06

These workers found the temperature coefficient to be an arithmetic function instead of the geometric function reported by the earlier workers. The mean of eleven experimental values was noted to be 2.21% per °C, with a standard deviation of 0.5. Truesdale and Vandyke[15] used channel methods similar to those used by Streeter, Wright, and Kehr, and obtained results of 1.018, 1.015, 1.008, and 1.018 for the temperature coefficient and concluded:

". . . until further, more conclusive, work is done on the subject the mean value of 1.5 percent, per °C, is the most suitable one to use for the temperature coefficient of the exchange coefficient in flowing water."

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13 "The Mechanics of Reaeration in Natural Streams," by D. J. O'Connor and W. E. Dobbins, Proceedings, ASCE, Vol. 82, No. SA 6, 1956, p. 115.

14 Journal of Applied Chemistry, A. L. Downing and G. A. Truesdale, Vol. 5, 1955, p. 570.

15 Water and Waste Treatment Journal, G. A. Truesdale and K. G. Vandyke, Vol. 7, 1958, p. 9.

Truesdale and Vandyke also found the temperature coefficient to be an arithmetic function.

In a recent review of the literature on this subject, in connection with low-flow augmentation studies, C. H. Hull and H. C. Carbaugh[16] state:

> ". . . it has been concluded that the value of θ should be rounded to 1.02. This conclusion should not be interpreted to mean that 1.02 is the correct value for θ or that there is no need for refinement or further study of this coefficient. It means only that refinement cannot be justified on the basis of the available evidence.
>
> With regard to the other conflict between a linear relationship between temperature and k_2 on the one hand, and a geometric relationship on the other, it is felt that for relatively small temperature corrections there is little to choose between the two without further experimental evidence. The two methods do not yield significantly different results unless a large temperature correction is involved. For large temperature changes, it appears that the available evidence favors the use of the linear relationship."

In view of the lack of agreement in this field shown by the cited literature, and the importance of the temperature coefficient in basic studies of stream reaeration rates, it was necessary to undertake experimental evaluation of the effect of temperature on k_2 under conditions that would be applicable to those found in natural streams.

PRELIMINARY INVESTIGATIONS

From the literature it appeared that two general techniques had been used for experimental evaluation of θ. Artificial channels have generally resulted in quite variable results which appear to be related to an inability to control all the pertinent factors (especially temperature) during the experiments.[11,15] On the other hand, the use of a fixed volume of water stirred at a constant rate appeared to provide much more consistent data.[6,14] A device was, therefore, designed that would allow for the careful control of water temperature, the degree of turbulence in the water, as well as the rate of flow, temperature, and the relative humidity of the air to which the water was exposed.

The original design incorporated a membrane electrode system as the detector element for the determination of intermediate concentrations of dissolved oxygen during the experimental period with initial and final D.O. determinations being used as calibration points for the electrode system. This system gave promise of providing the most comprehensive body of data available in this field in a relatively short period of experimentation. Extensive testing using this device indicated however, that two faults existed in the system.

1. It was found that the positioning of the electrode within the reaction flask was extremely critical and that the presence of the electrode had a

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16 Report No. VI, by C. H. Hull and H. C. Carbaugh, Low-Flow Augmentation Proj., Dept. of San. Engrg. and Water Resources, The Johns Hopkins Univ., Baltimore, Md., August 31, 1959.

large effect on the turbulence produced by the same degree of stirring. In fact, the k_2 of an experiment could be more than doubled by simply removing the electrode column from the reaction flask.

2. The other defect in this system, discovered in these preliminary experiments, was that under the specific conditions of use, the membrane electrode failed to give an exactly linear response to changes in D.O. concentration over the 5-hr to 10-hr test periods. This lack of linearity increased with the age of the membrane being used, from a hardly detectable amount during the first few hours to an easily discernable amount after a total of some 40 hr of use.

In view of these difficulties it was necessary to abandon the membrane electrode system and use instead the much more conventional technique described subsequently.

EXPERIMENTAL TECHNIQUE

The final technique used in the studies reported here consisted of determining the initial D.O. concentration of a fixed volume of partially deaerated distilled water and then exposing this water to the atmosphere under known conditions of temperature, turbulence, barometric pressure, and relative humidity for a measured length of time. At the end of the selected time interval, a final D.O. determination provided the data for the computation of k_2. The above process was repeated several times at each temperature and then the temperature changed and the entire series repeated. Four replicate dissolved oxygen determinations were completed at the beginning and end of each experiment by back-titration of an accurately measured excess of standard 0.005 N sodium thiosulfate to the amperometric endpoint. This process for D.O. determination has been shown to have a standard deviation of 0.007 mg per l.[17]

EQUIPMENT

Two of the major units of equipment for this study (amperometric titration device, and thermostatic water bath) have been described previously.[17] The general equipment layout can be seen in Fig. 1. The remaining units of equipment used in this study are described in the following:

Reaction Flask.—The container for the distilled water under test was constructed from an 18-liter borosilicate glass carboy to which two additional 35-mm inside diameter ports were added. These additional ports were equipped with standard ground-glass joints to allow entry of the various appurtenances described subsequently. Near the top of each port tube, a scribed line was used as the "upper calibration line."

1. Nitrogen Diffuser.—To partly deaerate the water, a glass tube, equipped with a fritted end, was used to disperse gaseous nitrogen throughout the water

17 "Solubility of Atmospheric Oxygen in Water," 29th Progress Report, Committee on San. Engrg. Research of the San. Engrg. Div., Proceedings, ASCE, Vol. 86, No. SA 4, 1960, p. 42.

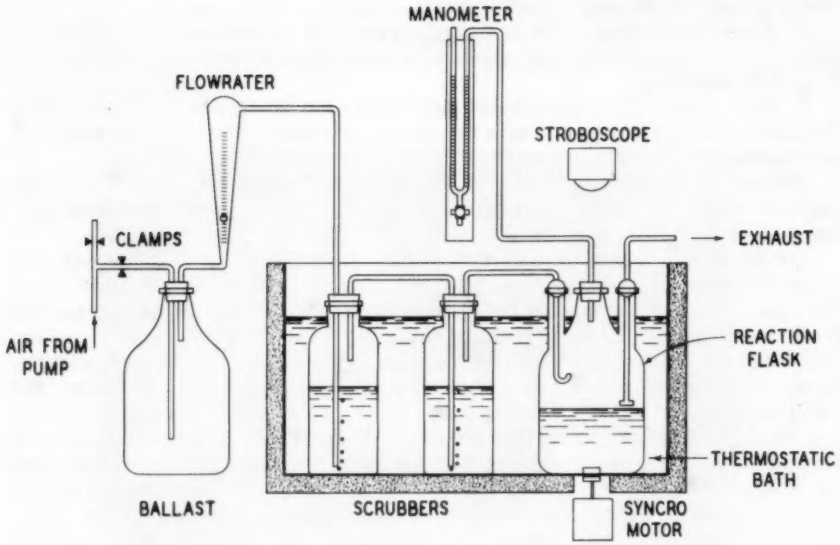

FIG. 1.—GENERAL EQUIPMENT ARRANGEMENT

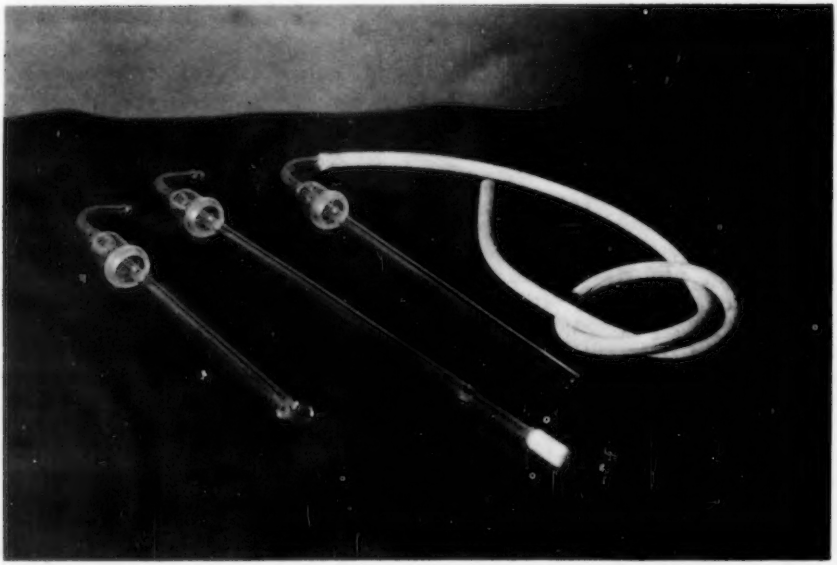

FIG. 2.—REACTION FLASK ATTACHMENTS

in the reaction flask. The nitrogen was supplied from a cylinder through a dual gage and needle valve arrangement.

2. Sampling Syphon.—The sampling syphon was constructed of 8-mm borosilicate glass, equipped with rubber tubing, to allow withdrawal of initial and final D.O. samples.

3. Exit Tube.—The air-exhaust tube was also composed of 8-mm inside diameter borosilicate glass, with the lower extremity being in the shape of an inverted "T."

These three units (illustrated in Fig. 2) were installed successively in the right-hand port of the reaction flask during the test procedure (described subsequently).

Air Supply.—Air was supplied by a conventional pump of 35 liters per min capacity, with the major quantity of the pumped air being bypassed and only 1 liter per min being passed on through the rest of the train. The air passed from the pump through an 18-liter ballast, through a flow-rating tube, and then was bubbled through two 9-liter bottles of distilled water immersed within the thermostatic water bath. The cooled and saturated air then entered the reaction flask through a J-tube that prevented even this small air flow (1 liter per min) from impinging on the surface of the water under test. The air was exhausted from the reaction flask through the T-shaped exit tube that provided an exhaust intake port as near the surface as reasonably possible. A manometer filled with water was continually attached to the air space in the reaction flask to determine whether the air flow caused an increase in the internal pressure above the barometric pressure of the laboratory. A readable differential was not detected during any of the test periods.

Magnetic Stirrer.—A 600-rpm synchronous motor was equipped with the magnet from a regular laboratory magnetic stirrer. This device was mounted under the stainless steel plate of the thermostatic water bath and drove a teflon-covered impeller centered within the reaction flask. The speed of rotation of the impeller was checked at regular intervals with a stroboscope. The impellers used were of two sizes:

1. 13 mm x 42 mm (large)
2. 7 mm x 24 mm (miniature)

Overflow Container.—The "lower calibration level" in the reaction flask was fixed by measuring the volume of water removed during the initial steps of each experiment. Four D.O. samples were removed in each case and the overflow from the D.O. bottles, plus any additional water necessary, was caught in the calibrated container shown in Fig. 3. To avoid any variation in the volume removed, including the samples themselves, the same four D.O. bottles were used in all determinations. A scribed line in the narrow neck of the overflow container served as a calibration point.

PROCEDURE

1. Adjust the mercury relay system to the temperature of the test and allow enough time for the thermostatic water bath to stabilize (usually overnight).

2. Fill the reaction flask to the upper calibration line with distilled water previously adjusted to the approximate temperature of the test. Allow sufficient time for the distilled water to assume the exact temperature of the test.

3. Read the mercury barometer and record the barometric pressure, and set the barograph.

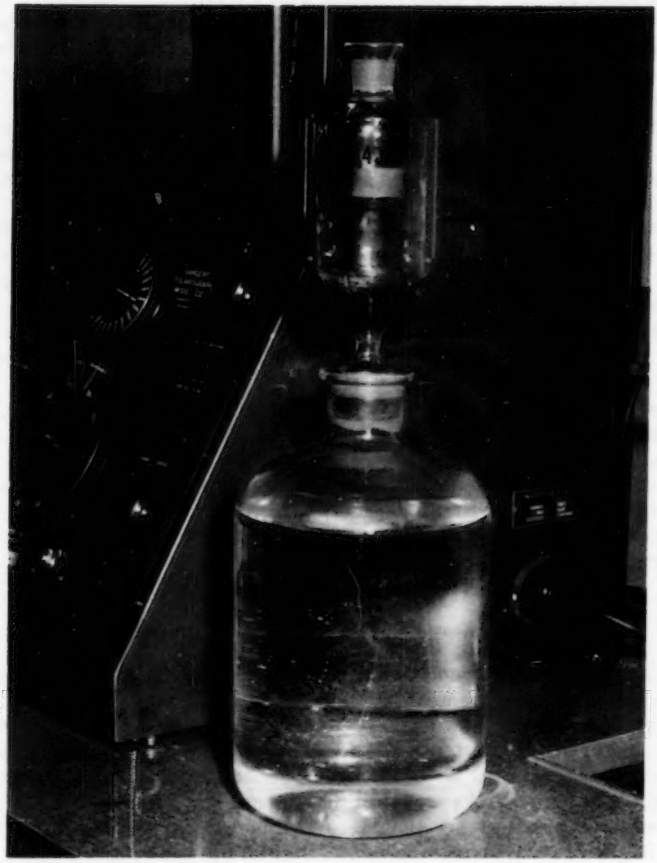

FIG. 3.—CALIBRATED OVERFLOW CONTAINER

4. Insert the selected magnetic impeller and start the synchronous magmetic stirrer.

5. Insert the diffuser tube into the right-hand port of the reaction flask, and deoxygenate for 1 and 1/2 hr with a stream of nitrogen. Remove the diffuser tube.

6. Check the level of the water in the reaction flask and readjust to the upper calibration line with temperature-adjusted, deoxygenated, distilled water (usually a matter of a few milliliters).

7. Determine the temperature to ± 0.01 °C of the water in the reaction flask and in the water bath.

8. Insert the syphon in the same port used previously for the diffuser and withdraw a fixed amount of water, ending with the collection of four replicate D.O. samples. Record the exact time of the collection of each sample. Rapidly syphon out the small amount of additional water required to fill the overflow container to the scribed line and remove the syphon. The water surface in the reaction flask is now down to the lower calibration line.

9. Install the air supply line in the left port and exit tube in the right port and adjust the air flow to 1 liter per min. through the reaction flask area above the distilled water. Connect manometer to reaction flask center port and check for zero differential pressure.

10. Allow stirring to proceed for the predetermined period of time that will allow a D.O. pickup of more than 1 mg per liter, but will not approach closer than 1 mg per liter to D.O. saturation (usually 2 hr to 4 hr, depending on the impeller and the test temperature).

11. At the exact end of this period, disconnect the air supply and exit tube. Carefully insert the syphon and withdraw four replicate D.O. samples while recording the time of collection of each.

12. Read and record the water temperature in the reaction flask, and in the water bath.

13. Read and record the barometric pressure and barograph chart.

14. Determine the D.O. concentration of the four initial and four final samples by the amperometric titration technique.

15. Compute k_2 for the period of time extending between the last of the initial and the first of the final D.O. samples.

RESULTS

Large-Impeller Experiments.—Fifteen experiments were completed with the large impeller, with temperature at 10°C, 20°C, and 30°C, using an exposure period of either 2 hr or 4 hr. Comparison of the data of the two different time intervals indicated strict adherence to the Adney-Becker law (reaeration rate proportional to the deficit) under the experimental conditions being used. The level of k_2 found in these tests ($k_2 = 5\pm$ at 10°C, to $k_2 = 8\pm$ at 30°C) was somewhat higher than usually found in natural streams. In addition, a degree of vortexing was noted to occur in the reaction flask during the test period. For these reasons, the data from the large-impeller experiments were not used in the development of the final value of θ. The results obtained have been useful, however, in indicating little change in θ over a wide range of turbulence conditions.

Miniature-Impeller Experiments.—In order to adjust the rate of reaeration to a level commensurate with that found in natural streams and to provide a more constant interfacial area by the elimination of vortexing, a series of 18 experiments was undertaken under conditions identical to those previously described except that a miniature impeller, driven at the same speed, replaced the much larger one previously used. As was mentioned earlier, when using

the large impeller, both 2-hr and 4-hr exposure periods were used. With the miniature impeller, however, it was necessary to use at least a 4-hr exposure period to obtain a pickup greater than 1 mg per liter D.O. so as to insure a high-precision determination of the difference in D.O. levels. Triplicate measurements of k_2 were completed at 5°C, 10°C, 15°C, 20°C, 25°C, and 30°C. Complete freedom from vortexing was obtained at this lower turbulence level and, in addition, a higher degree of precision was obtained.

The data derived from these 18 experiments are shown in Table 1, and in plotted form in Fig. 4. Inasmuch as these points fall along a straight line on

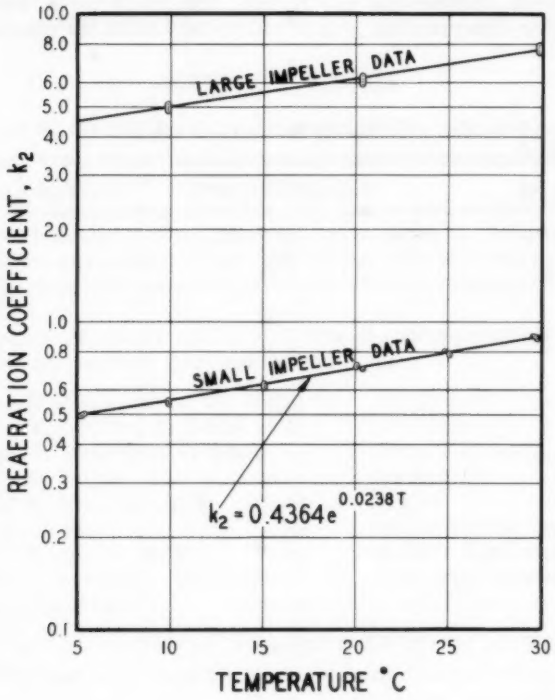

FIG. 4.—EXPERIMENTALLY DETERMINED EFFECT OF TEMPERATURE ON k_2

semi-log paper (and definitely do not form a straight line on rectangular-coordinate paper), the plotted values were fitted by the least-squares technique, using the IBM-704, to an equation of the form

$$k_2 = ae^{bT} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

in which a is the intercept constant, b is the rate constant, and T is temperature in °C. After the constants are inserted from the computer calculation,

Eq. 5 becomes

$$k_2 = 0.434\, e^{0.0238\, T} \quad \ldots\ldots\ldots\ldots \quad (6)$$

which can be converted to the more familiar form

$$k_2(T°) = k_2(20°) \left[1.0241^{(T-20)}\right] \quad \ldots\ldots\ldots \quad (7)$$

Comments.—Both series of experiments are shown in Fig. 4. The similarity of slopes of the two least-squares lines of best fit is indicative of rather good agreement between the two series. The value for θ, derived from the large-impeller experiments, was found to be 1.0226, as compared to 1.0241 for the small-impeller experiments. The correlation coefficient of the experimental data for the earlier experiments was 0.9935, while this value was raised to 0.9978 by the improvements in experimental conditions of the later experiments. Consequently, the results of small-impeller experiments have been accepted as providing the best value for the temperature coefficient.

TABLE 1.—EXPERIMENTAL DETERMINATION OF θ

Test No.	Temperature, in °C	Reaeration Coefficient k_2
1	5.27	0.5019
2	5.19	0.4973
3	5.19	0.4972
4	9.95	0.5544
5	9.95	0.5496
6	9.99	0.5424
7	15.06	0.6257
8	15.06	0.6082
9	15.04	0.6304
10	20.36	0.6974
11	20.08	0.7096
12	20.10	0.7143
13	25.06	0.7876
14	25.06	0.7796
15	24.85	0.8064
16	29.87	0.8918
17	29.88	0.8830
18	29.66	0.8989

The fact that the data establish a straight line on semi-log paper is, of course, indicative of a geometric, or constantly proportional, increase of k_2 with temperature. This is in agreement with earlier workers but differs from

the conclusions drawn by Downing and Truesdale[14] and Truesdale and Vandyke.[15]

CONCLUSIONS

A substantial degree of confusion exists in the literature regarding the value of the temperature coefficient θ of the reaeration coefficient k_2. A series of carefully controlled experiments is reported here that indicates θ to be 1.0241. This coefficient remains substantially constant over a wide range of turbulence conditions. Thus, the rate of reaeration has been shown to increase at the geometric rate of 2.41% per °C, throughout the temperature range found in natural streams.

ACKNOWLEDGMENTS

This report was prepared by Harold L. Elmore, Public Health Engineer, and William F. West, Analytical Chemist, Stream Sanitation Staff, Division of Health and Safety, Tennessee Valley Authority.

The Sanitary Engineering Division gratefully acknowledges the generosity and professional courtesy of the authors of this paper in making the results of their research work available for publication. The SED Research Committee also recognizes the cooperation of the TVA in releasing this paper for publication here.

This research report is one of a series of professional contributions by The Committee on Sanitary Engineering Research:

E. R. Hendrickson, M. ASCE	Air Pollution
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M. A. Churchill, M. ASCE, Head

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Proceedings of the American Society of Civil Engineers

UPFLOW SOLIDS CONTACT BASIN

By A. W. Bond[1]

SYNOPSIS

Considerations of methods of separating the solids from the water treatment processes and the value of "solids contact" are presented herein.

The design of a simple type of upflow basin is recommended and examined and an example of the design of such a basin built to clarify, or soften or both, the water supply of the town of Bacchus Marsh, Australia by means of the lime-soda process is given.

One of the principal features of this basin is a new device—a slurry weir—for transferring surplus old solids from the top of the slurry pool to the concentrator at the same rate as new ones are formed at the bottom of the slurry pool. By this means, the level of the top of the slurry pool or sludge blanket is automatically held at a constant level, thus obviating the danger of decanting slurry with the treated water.

The quantity of slurry produced under varying conditions and its concentration in the upflow zone and concentrator are calculated, and the volumes and areas required to handle it are examined.

Note.—Discussion open until April 1, 1962. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SA 6, November, 1961

[1] Cons. Engr., Bacchus Marsh, Australia.

The method of decanting the supernatant liquor is described, and a brief summary of the performance of the basin since it commenced operation in August, 1959, is given.

INTRODUCTION

The most common method of removing impurities from water is to add to it one or more soluble chemical compounds which combine chemically with impurities or harmless compounds already in solution, in either case forming insoluble precipitates which adsorb suspended impurities. The precipitates are then separated from the water.

The added chemicals must first be dissolved in the water, when the chemical reactions immediately occur. Time must then be allowed to enable the new insoluble compounds to precipitate out and grow into large particles.

If the proportion of precipitated solids to water is relatively low, filtration alone will suffice to remove them. However, if the proportion is high, preliminary separation of the bulk of the solids by settling is necessary.

Because large particles settle more rapidly than small ones, separation is facilitated if large particles can be induced to grow. They are usually flocculent, with a density only slightly greater than that of water.

The particles will form most readily and grow most rapidly in the presence of abundant nuclei in a supersaturated solution of the same chemical composition. In many cases agitation assists the process. If sufficient time is permitted, precipitation of the solids is carried almost to completion.

If solids contact, as this process is called, is included in the water treatment process, time will be saved with consequent economy in the compartment through which the dosed water is passed while the particles form and grow.

The conventional method of pre-treatment is to pass the suspension of solids and water horizontally and longitudinally (sometimes radially) through a sedimentation basin.

A long retention period (4 hr to 6 hr) is necessary to ensure adequate separation. In addition, 30 min to 50 min are usually allowed for conditioning the precipitates. The plant required is relatively large and costly.

The long retention period discourages the use of solids contact by sludge return, because disagreeable tastes and odors are formed, especially when the water is polluted or carries algae.

The great bulk and area of water is conducive to the formation of density and wind currents, and the efficiency of the basin suffers accordingly. Even by the judicious use of baffles it is difficult to obtain uniformity of velocity distribution under varying conditions, and severe short circuiting is a common defect.

The size and shape of such a basin requires the use of a nearly horizontal floor, necessitating expensive mechanical equipment for effective sludge removal.

In practice, the performance of horizontal flow basins is highly variable, and in some cases is consistently poor.

During the last few decades of the nineteenth century, an upward flow basin was developed in Great Britain and on the continent of Europe which, in its best form, overcomes all these difficulties and gives consistently good results.

Since the first invention, many variants have been tried. All led to greater complexity and nearly all to lower efficiency. This and the fact that the "improvements" were patented, discouraged their use, and the horizontal flow basin continued to be built.

Upflow basins have become esteemed in the United States and several efficient and economical types are in use, but all are more complex than the type to be described.

It is, of course, a matter of opinion as to what is the best form of basin. It is here submitted that the original simple type still used by British and European engineers is the most efficient and economical.

In this type, the water flows upwards through an inverted pyramidal shaped basin. Except for a few feet at the top, the basin is filled with previously precipitated solids accumulated in a dense mass called a slurry pool or sludge blanket. Through this the water percolates upwards, giving the desired solids contact, and is decanted from the clear water zone at the top.

The formation, and to a lesser extent the growth of particles, is a slow process, even with the aid of solids contact.

Under the most propitious conditions, solids contact should continue for at least 1 hr. A conservative design would provide for 1½ hr.

Allowing for the time occupied in mixing the chemicals and in rising through the clear water zone, the retention time is about half that required for horizontal flow basins.

The time required for the growth of particles into floc in the conventional type of basin is eliminated in the upflow basin, as flocculation occurs concurrently with particle formation.

No added separation time is required. The shape of the basin, the direction of flow, and the presence of the slurry pool, with certain minor safeguards, ensure complete separation of the water from the precipitated solids as it leaves the top of the slurry pool.

The short retention period reduces space and capital requirements; separate basins for conditioning the floc and for separation are not required; the chemical reactions are carried to practical completion; tastes and odors have less time in which to develop; density currents are inhibited; and the small surface area reduces susceptibility to wind currents. Short circuiting can almost be eliminated, as relatively uniform dispersal of the water at entry and withdrawal may be achieved. In addition, sludge removal is simple.

Notation.—Letter symbols adopted for use in this paper are defined where they first appear and listed alphabetically for ease of reference in the Appendix.

Crystallization.—In a supersaturated solution, the excess solute may normally be expected to crystallize into the solid form, leaving the solution saturated.

Certain substances, however, including some of those formed in water treatment processes, exhibit an habitual reluctance to crystallize, and in such cases it is a matter of routine to "sow" or "inoculate" the refractory solution with crystals of the solute in question.

The particles of the solid phase when first formed, at the moment of solution of the reagents, are of molecular dimensions, and are stated by some authorities to be crystalline for all substances.

If old crystals are present, the new reaction products are deposited directly on the surfaces of the old solids, saving part, sometimes a considerable

part, of the solute-solids separation time. The time, which would otherwise have been required for spontaneous crystallization, is almost completely saved.

The rate of crystallization may be increased by (a) increasing the number and decreasing the size of the old crystals, and so increasing their surface area, and (b) decreasing the thickness of the diffusion layer, and so increasing the rate of diffusion from the bulk of the solution to the old crystal faces. Stirring will reduce the thickness of the diffusion layer, but would have to be extremely vigorous to eliminate it altogether.

The compounds which are precipitated are not absolutely insoluble. It is impossible to reduce their concentration below the solubility limit, and it is often difficult in practice to reduce the concentration to this limit. Large particles have a lower solubility than small ones. Hence, the larger the particles that can be formed, the greater the proportion of the solute that will be precipitated and can be removed.

It also follows that, if a precipitate initially contains particles of different sizes, a solution which is saturated with respect to the larger particles will be undersaturated with respect to smaller ones, and the latter will begin to dissolve. In consequence, the solution will become supersaturated with respect to the larger particles, and fresh material will be precipitated on them. Such a process will finally lead to the disappearance of the smaller particles. If it is desired to encourage this process, the precipitate should be disturbed as little as possible.

Coagulation or Flocculation.—The process by which the minute primary particles of a sol or a solution become aggregated together in a non-crystalline fashion by collision and adhesion and form floc is termed coagulation or flocculation. In water treatment processes either term is loosely used to include all the processes that take place, from the addition of the chemicals into the raw water to the formation of the large floc particles. Some of the particles originally present, as in waters with a high organic content, or formed in the treatment process, as in softening water high in magnesia, are so small as to approach the colloidal state. The particles carry an electrical charge, repel each other, and will not aggregate. The addition of a coagulant whose particles carry an opposite charge results in mutual neutralization, and the particles then readily aggregate. Other small particles without charge are adsorbed on or entrained in the floc.

Although the coagulum or secondary aggregate is non-crystalline, the sol particles themselves may be either crystalline or non-crystalline.

A dense accumulation of floc particles held in suspension is known as slurry. Settled slurry is known as sludge. Although there is no clear line of demarcation between slurry and sludge, a convenient arbitrary division is the packed point.[2]

Solids Contact.—From the foregoing, it is apparent that the ideal solids contact process is one so arranged that the dosed water is first brought into intimate contact with a relatively large quantity of very small old particles at a high concentration and in rapid movement and, subsequently, with a large quantity of the largest possible old particles at high concentration and in a state of quiescence. In such a process, the presence in the early stages of large numbers of nuclei will encourage the solute, in a supersaturated solution, to crys-

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[2] "The Behaviour of Suspensions," by A. W. Bond, Proceedings, ASCE, Vol. 86, No. SA 3, May, 1960, p.57.

tallize out as a solid precipitate; in the later stages, in the presence of the large particles, precipitation of the solids will be conducted nearly to completion.

Provided that sufficient time is allowed, the concentration of solids remaining in solution will be reduced nearly to the saturation point, and the solution will be stable for the existing temperature and hydrogen-ion concentration.

Solids contact may be provided by slurry recirculation, that is, by passing previously precipitated solids back to the dosing point, or by passing the water through a slurry pool consisting of a large dense accumulation of previously precipitated solids.

Slurry Recirculation.—It was established by the early authorities and has been repeatedly confirmed that, if coagulants are added an appreciable length of time before the dosed water is brought into contact with old slurry, the new precipitates may and probably will be small and inefficient and fail to adhere to the old particles.

In the horizontal flow basin, it is often necessary, for this reason, to add recirculated slurry to the water before or immediately after it is dosed. The presence of recirculated slurry in the flocculation zone is essential. Yet, recirculation of slurry in such basins is often undesirable or impossible because the long period of contact between water and slurry intensifies the tastes and odors picked up by the water.

Slurry recirculation does not fulfill the conditions prescribed for ideal solids contact. The quantity of slurry recirculated must necessarily be low in proportion to that of the water, and the concentration will, therefore, be low. No provision is made for subsequent contact with large particles.

The Slurry Pool.—It is generally accepted that a much more satisfactory method of providing solids contact is to pass the dosed water through a dense accumulation of slurry in an upflow basin where it is more intimately mixed with slurry of a higher concentration than it could be in a horizontal flow plant. The quantity of slurry relative to the rate of flow of the water may be made as large as desired.

In order to pass the water through the slurry the latter must be held in suspension. This may be done by mechanical or hydraulic means. In either case much the same amount of energy must be expended.

The best way of passing water through a suspension of large particles in a state of quiescence is to use the water itself to suspend the particles by causing it to flow gently upwards between them.

The Upflow Zone.—It has been shown that a suspension in an upflow zone with vertical sides is inherently unstable, and that, on the other hand, a stable slurry pool can readily be formed and maintained in a uniformly expanding upflow zone.

In such a zone, random concentrations of particles higher than average settle on the sloping side walls and roll down them, until they reach a level where the area is so restricted and the velocity so high that they are again lifted into suspension.

The same effect is produced by wall drag, which reduces the upflow velocity near the walls and, thus, permits the particles to settle down them.

Large particles reaching the bottom are broken up, and swirl up the center with an initially high but gradually decreasing velocity. A definite circulation is created in the lower portion of the slurry pool, but the greater part at the

top shows no translatory movement, only the quiet twirling of the particles under the influence of the upflowing water.

A slurry pool in such an upflow is, thus, inherently stable and automatically corrects any variations in concentration at each level. Any desired concentration may be achieved by selecting the appropriate dimensions.

When the dosed water is first introduced into such a zone, precipitates form and gradually accumulate to form a slurry pool, which is then retained in use for long periods. Excess solids must be removed to retain the desired volume and concentration, and to maintain the top at the desired level. In the result, the precipitates composing the slurry pool are continually discarded and replaced, although the pool itself remains in being.

Suitable conditions for solids contact are thus attained. After first coming into contact with a large quantity of small particles in a state of rapid movement the dosed water then, through gradually changing conditions, encounters successively larger particles, moves at decreasing speed, and finally reaches a zone of very large particles at high concentration and in a state of quiescence.

Moving parts and mechanical appliances involving repair and maintenance work in inaccessible positions are eliminated, as the slurry pool is kept in suspension without mechanical agitation, and the hopper shaped bottom facilitates hydraulic removal of sludge as necessary.

An incidental but extremely valuable property of a slurry pool suspended in an expanding upflow zone is the dispersing effect of the particles on the water, which can be made to rise with practically uniform velocity over the entire area of the pool by providing satisfactory decanting arrangements.

Control of Pool.—Proper separation depends very largely on maintenance of the top of the slurry pool at the correct elevation, so that, although maintaining sufficient height and density to give adequate solids contact, it will not decant with the supernatant liquor.

Since 1952, a new device called a slurry weir has been developed at Bacchus Marsh, Australia that enables the desired control to be simply and surely attained. Its construction and advantages will be explained herein.

EXAMPLE OF DESIGN

It is proposed to illustrate the principles described previously by applying them to the design of a small clarifying and softening upflow basin at Bacchus Marsh.

The basin, shown schematically in Fig. 1, consists of three hopper bottomed (inverted pyramidal) cells SBS in line, opening into the bottom of a single rectangular cell CD.

The water is dosed and mixed with the appropriate chemicals in a small mixing chamber (not shown), and introduced into the bottom of the solids contact zones SS by means of central downdraught tubes FF. It then flows upwards through the twin slurry pools, leaves the surface of the latter at the level CC, passes upwards through the decanting zone CD, and is decanted at the level DD, from which it passes to the filters.

Excess slurry spills over the slurry weirs at AA and settles into the concentrator cell B, from which it is removed as necessary to the drying beds. A general view of the basins is shown in Fig. 2.

Output.—The output of settled water required, after making due allowance for filter wash water, is 655 gpm spread over 22 hr on the day of maximum demand.

The plant is operated at a constant output, hourly fluctuations in the demand being balanced by providing an adequate reservoir of filtered water.

It was expected that the plant would be capable of operating at a higher output than this, and the results reported below confirm this. All conduits were designed for easy hydraulic conditions at an output of settled water of 983 gpm.

Quality of Water.—The raw water is usually turbid occasionally up to 500 ppm or more, and the color is often 100 ppm to 150 ppm. The hardness varies usually from 60 ppm to 300 ppm, the highest recorded value being 550 ppm.

Method of Treatment.—It is proposed that whenever the hardness exceeds 75 ppm to 80 ppm the water will be softened by the lime-soda process to 60 ppm to 70 ppm, separating out the solids in an upflow basin and by filtration. Tests with a model[2] showed that turbidity and color would be effectively reduced by this process for this particular water, and this has been confirmed in operation of the completed plant.

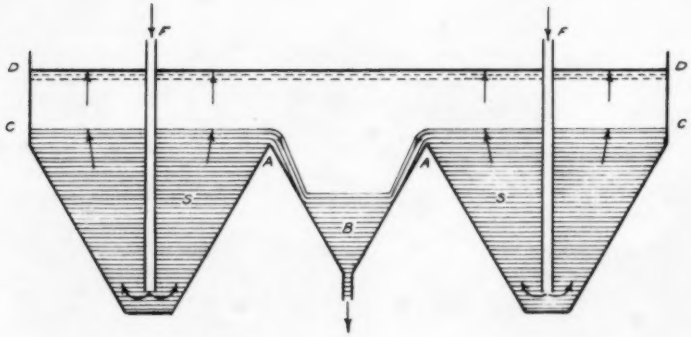

FIG. 1.—DIAGRAMMATIC SECTION OF DOWNDRAUGHT TUBES, UPFLOW BASINS, SLURRY WEIRS, AND CONCENTRATOR

If the hardness falls below 75 ppm, turbidity and color will be reduced by plain coagulation, separating out the solids as when softening.

Quantity of Slurry Produced.—The slurry produced by plain clarification plants is not very considerable, and its handling and disposal does not usually constitute a difficult problem.

Lime-soda softening plants produce large quantities of slurry, and special arrangements must be made for concentrating, handling and disposing of it.

From the analysis of the water, the theoretical dry weight of the reagents and of the reaction products for the lime-soda process may be calculated.

A simple method of doing this is shown in Table 1, which gives the chemical reactions that occur, the reagents required for each kind of hardness, and the insoluble products of the reactions. The table also shows the dry weight in parts per million of the required dose of each reagent and of the resultant amount of each insoluble product, calculated from the analysis.

In the table, hydrated or slaked lime, $Ca(OH)_2$ is shown as a reagent. The equivalent quantity of quicklime, CaO, may be obtained by multiplying the quantity of slaked lime by 0.75. Allowance must, of course, be made for impurities in commercial reagents in calculating the dose.

Not all of the insoluble reaction products can be removed from the water. It is possible to reduce their concentration nearly to the solubility limit, but in practice, for municipal purposes, it is not economical to reduce the total hardness below about 60 ppm - 70 ppm. Hence, the precipitates removed from the water will be 60 ppm - 70 ppm less than the calculated quantity of the insoluble reaction products.

The Wet Solids.—These precipitates will appear in the water as floc, with a low settling velocity.

The concentration of the slurry—the accumulation of floc particles in suspension—is most conveniently measured by the solids fraction, s, which is expressed as a decimal fraction.

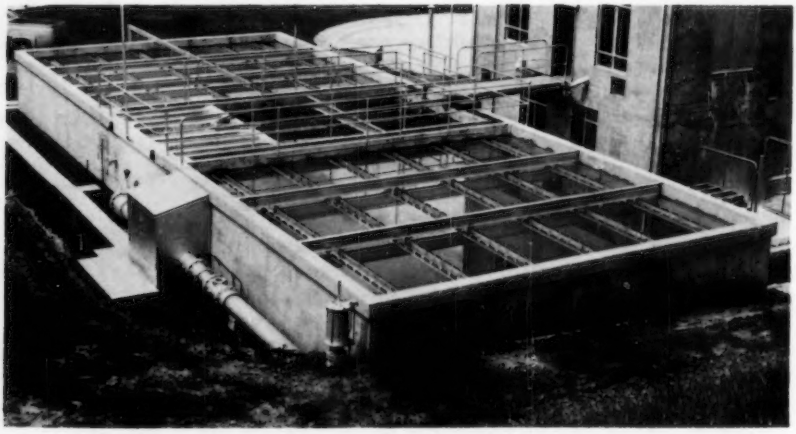

FIG. 2.—MIXING BASIN, SEDIMENTATION BASINS AND CONCENTRATOR

The solids fraction is the ratio of the final volume of wet settled solids to the original volume of the slurry.[2]

The solids in such a slurry will at first settle with a constant velocity until they reach the concentration known as the packed solids fraction. They will then compact slowly to their final wet settled volume and become sludge.

A sample of lime slurry at the packed solids fraction will be found, for this water, to have a solids fraction of 0.30.

A sample of sludge (the wet settled and compacted solids) will, of course, have a solids fraction of unity.

It was proposed, in this plant, to settle the slurry in an internal concentrator to the packed solids fraction, and then to withdraw it and allow it to compact in external sludge beds, where it would be partially dried after the supernatant liquor from the drying beds was returned to the filters. Actual practice has not followed this pattern, as will be explained subsequently.

Throughput and Slurry Removal.—Under the worst recorded conditions at Bacchus Marsh, when the hardness of the water was 550 ppm, the following analytical determinations were made:

Calcium Hardness = 160 ppm, Magnesium hardness = 390 ppm and Alkalinity (bicarbonate) = 140 ppm. Free carbon dioxide is neglected for the present purpose. Following Table 1, $CaCO_3$ produced = (0 + 550 + 140) = 690 ppm $Mg(OH)_2$ produced = 390 x 0.58 = 227 ppm. Total weight of dry solids produced = 917 ppm. If 60 ppm of these solids are left in the water, then the total weight of dry solids removed = 917 - 60 = 857 lb per 100 Imperial gal.

One cubic foot of wet sludge, at unit solids fraction, of $CaCO_3$ and $Mg(OH)_2$ in the preceding proportions will weight about 64.5 lb. This volume of wet solids, when dried, will weight about 7.5 lb.

TABLE 1

Reactions

$CO_2 + Ca(OH)_2 = CaCO_3 \downarrow + H_2O$ . . . . . (1)

$Ca(HCO_3)_2 + Ca(OH)_2 = 2\,CaCO_3 \downarrow + 2H_2O$. . . . . (2)

$Mg(HCO_3)_2 + Ca(OH)_2 = CaCO_3 \downarrow + MgCO_3 + 2H_2O$ . . . . . (3)

$MgCO_3 + Ca(OH)_2 = Mg(OH)_2 \downarrow + CaCO_3$ . . . . . (4)

$MgSO_4 + Ca(OH)_2 = Mg(OH)_2 \downarrow + CaSO_4$. . . . . (5)

$CaSO_4 + Na_2CO_3 = CaCO_3 \downarrow + Na_2SO_4$. . . . . (6)

Doses

Lime - ppm $Ca(OH)_2$ = (CO_2 + carbonate hardness + magnesium hardness) x 0.74

Soda-ash - ppm Na_2CO_3 = (Non-carbonate hardness-X) x 1.06 where X is the non-carbonate hardness to be left in the water.

Products

$CaCO_3$, ppm = CO_2 + total hardness + bicarbonate hardness

$Mg(OH)_2$, ppm = Magnesium hardness x 0.58

(N.B. All terms on the right hand side of the equations for calculating doses and products are expressed as ppm $CaCO_3$).

Hence, every 1000 gal of this quality water, when treated, will contain $\frac{8.57}{7.5}$ = 1.15 cu ft of wet settled solids at s = 1.0, which will weight 1.15 x 64.5 = perhaps 75 lb. This is equivalent to $\frac{1.15}{0.3}$ = 3.8 cu ft, or 24 gal of wet slurry at the packed solids fraction $s_P = 0.3$, the concentration at which it was expected to be withdrawn. A thousand gallons of treated water would, thus, produce 976 gal of settled water.

Thus, the throughput of the upflow basin would be 671 gpm for an output of 655 gpm, and 1007 gpm for an output of 983 gpm.

For throughputs of 671 gpm and 1007 gpm the amount of slurry removed at $s_P = 0.3$ would be 16 gpm and 24 gpm respectively, which are respectively

equivalent to 50 lb per min and 75 lb per min of wet settled solids at unit solids fraction. These last figures are the quantity P, the rate of removal in pounds per minute of wet settled solids at unit solids fraction. It is important to note here that the rate of removal in cubic feet per minute of wet solids at s = 1.0 is P/w and of slurry at any concentration s is P/(w s).

If all the slurry at $s_P = 0.3$ were run to waste, the loss would be about 2.4%. If the slurry is allowed to settle for 24 hours, and the supernatant liquor returned to the plant, the net loss from slurry removal, with the hardest water likely to be encountered, will be about 0.7%.

Addition of Reagents.—The efficiency and economy of a water treatment plant is completely dependent on the correct dosing of reagents into the raw water. Accurate and reliable appliances must be used for measuring, dosing and mixing. The chemical house should be properly and conveniently laid out, with ample storage and adequate handling facilities, so that the operating staff can conduct its work properly.

The most economical quality of chemicals, free of undesirable impurities should, of course, be used; that is, the quality of which the cost per pound of active reagent, delivered into the water, is least. Taking into account the cost of handling, storage, and dosing, this usually means that the best and purest quality is the cheapest. The chemicals should be finely divided to facilitate rapid solution.

The best results are obtained when the reagents are first dissolved or suspended in a relatively small amount of water and the solution or suspension is added to the raw water. Modern measuring machines, or "chemical feeders," usually provide this facility.

The Mixing Basin.—The function of the mixing basin is to disperse the dose of chemicals rapidly and uniformly throughout the incoming water and to ensure that it is dissolved.

No rules have been established for determining the shape and dimensions of a mixing basin, although a retention period of 2 min to 8 min is usually advocated. It is obvious that it should be small for the sake of economy in space and cost and to prevent premature crystallization. When chemicals already in solution are to be mixed with the raw water, a few seconds of turbulent flow in a pipe or channel will ensure complete dispersal, and no mixing basin need be provided. Only when the chemicals are in suspension is there any necessity for time and the means to be provided to put them into solution, and both of these will depend on the size of the particles.

In the case under consideration, the space occupied was of no account, and economy was achieved by using the walls of the upflow and concentration compartments.

Turbulence in the mixing basin greatly aids rapid solution and uniform dispersal of the chemicals. There are many standard methods of achieving it. If sufficient head is available, the energy of the incoming water may be used by jetting the water or by passing it through tortuous passages. Both have been used at Bacchus Marsh.

In some cases, turbulence is secured by means of motor driven paddles or by jets from a pump. The power required is about one quarter to one third horse-power per million gallons per day. Provision has been made at Bacchus Marsh to add additional jets, but they have not been found necessary. Such obvious precautions should be taken as the avoidance of dead pockets in which

silt, precipitates, or undissolved chemicals may accumulate, and the provision of access for painting and maintenance.

If mixing is incomplete flocculation may not be satisfactory. In underdosed portions of the water incompletely formed floc may result, and in overdosed portions too light floc may result.

It is true that these defects are much less likely to occur when the dosed water is passed through a suspended slurry pool, which ensures thorough mixing and solution. Any undissolved particles, being dense and compact, will circulate in the lower portion of the upflow zone until dissolved. This fact should not be taken advantage of in the design, but should be regarded as an additional margin of safety.

In the Bacchus Marsh plant, the mixing basin consists of a series of narrow channels connected by holes at the bottom which is rounded to give the mixture a tumbling or rotary motion. Quite large floc is formed in this basin, which has a theoretical retention period at the design flow through rate of 4 min. The basin is acting as a rolling mix flocculator, although the retention period is much less than is usually provided for flocculation. This was not intended, because formation of floc before solids contact is not desirable and could be harmful with other water and treatment. The conditions of flow from the mixing basin to the slurry pools are not such as would damage the floc, but in this particular case, it would not matter anyway, as the floc will re-form no matter how it is treated.

The Downdraught Tube.—During a plant shut-down, the solids settle to the bottom of the upflow zone and compact. On restarting, the compacted sludge must be broken up before it will lift into suspension. To do this, each downdraught tube has been provided with a nozzle so that the dosed water emerges in a jet, directed vertically downwards to the center of the floor of the upflow zone, as shown in Fig. 3.

The jet impinges on the apex of a steep sided pyramid built on the floor, with smooth deflecting curves at the base, so that a stream of water flows radially outwards along the floor. This stream and that of the jet will, on starting up, undermine and collapse the sludge that would otherwise cling to the walls and corners, and maintain, during operation, sufficient turbulence in the lower part of the upflow zone to ensure thorough mixing of the dosed water and slurry.

Two 8-in. diameter nozzles have been provided, one in each basin, which for a throughput of 671 gal per min, gives a nozzle velocity of 2.57 fps.

The total loss of head (including velocity head) in the mixing basin and downdraught tubes is 0.40 ft at the design throughput and with clean tubes. Chemicals gradually deposit in the tubes and increase the head so that every few weeks the tubes have to be cleaned. The deposit is soft and easily removed with a rubber disc on the end of flexible cleaning rods.

The extra head loss must be allowed for when providing freeboard in the mixing basin; so also must the extra head of water required to balance the dense slurry in the pool. This is only significant on starting up after the slurry has been allowed to compact.

The downdraught tube should be arranged, if possible, so that no part of it lies at a flatter slope than 1.75 vertical to 1.0 horizontal, because otherwise solids will collect on its upper surface and may ferment. If this is not possible, any flatter surface should be given a solid capping of some lightweight, but permanent material to provide the necessary slope to shed sludge.

Period of Solids Contact.—As soon as the reagents are dissolved the chemical reactions occur almost instantaneously, and the desired insoluble compounds are formed, but are, at first, in solution.

As already stated, the crystallization of the solute into solid particles is slow and the coagulation of the small particles into the non-crystalline floc only relatively less so. It is essential, therefore, that adequate time for solids contact between the incoming dosed water and the previously precipitated slurry should be allowed, to ensure that crystallization is as complete as economically possible. Provided sufficient time is allowed for crystallization, then ample time is concurrently available for flocculation.

It has long been known that 2 hr retention in conventional basins is sufficient for crystallization and flocculation, provided that recirculation of slurry is

FIG. 3.—DOWNDRAUGHT TUBE AND DEFLECTING PYRAMID

adequate and that short circuiting is slight. Longer retention periods are adopted largely because of failure in achieving these essential conditions.

If the reagents are completely dissolved, and the dosed water is then passed through and intimately mixed with previously precipitated solids in a stable slurry pool free from short circuiting, adequate crystallization and flocculation may be achieved in 1 hr contact time, or less, depending on the character of the raw water and the treatment process involved.

The Solids Contact Zone.—This is the zone of upflowing water in which the large body of slurry is held in suspension—the "slurry pool" or "sludge blanket"—through which the dosed raw water passes and so obtains the necessary solids contact.

The solids contact zone should lie almost entirely in the expanding upflow zone to ensure stability, but it may at times during operation encroach on the decanting zone above, which also contains upflowing water and which need not necessarily be expanding.

Shape.—Either an inverted cone or pyramid fulfills the condition of uniform expansion. The pyramid requires more material, but it is to be preferred to the cone, as its plane surfaces are much simpler to construct, and the total cost is usually lower. It also occupies less room and is more easily fitted to other parts of the plant.

Wall Slopes.—Experience has shown that the slope of the walls should be between 1.65 and 1.75 vertical to 1.0 horizontal.

If the slopes are flatter, the slurry will not roll down them, especially at the corners, and the upflow velocity at the surface may not be uniform, because of too rapid divergence of the walls.

If steeper, the zone will be more uniform in shape, and less stable. The greater depth necessary will increase the cost.

Let d be the depth and b the width at the (square) top of the solids contact zone. Then d = 0.875 b.

The bottom should be truncated to form a square base, sufficiently large to give the bare minimum room for cleaning and painting. If the base is too large, the jet from the downdraught tube will not prevent the accumulation of sludge, with its concomitant tastes and odors.

Volume.—The volume of the solids contact zone must be such that the desired time of contact is attained while the maximum throughput is traversing the zone.

For the present design, a conservative view was taken, and 1 ½ hr solids contact time provided. Any reduction in this time that may be found possible will permit the plant to operate at a higher throughput than the design value. (Experiments with the completed plant have shown that the time of solids contact can be greatly reduced, as will be reported.

Let Z be the volume of the solids contact zone, in cubic feet; t the time of solids contact, in minutes; and Q_1 = throughput, in gallons per minute. Then

$$Z = \frac{Q_1\ t}{6.23} = \frac{b^2\ d}{3} = 0.292\ b^3 = 0.437\ d^3 \quad \ldots\ldots\ldots (7)$$

In the present design, Q_1 = 336 gpm throughput per basin, and t = 90 min. Thus, b = 25.5 ft, d = 22.3 ft, and Z = 4850 cu ft.

The apex of the pyramid has been truncated to provide a bottom 5 ft square. In addition, the dimensions have been adjusted to b = 26 ft and d = 17.5 ft to give the same volume.

Subdivision and Depth.—Deep, narrow excavations and construction work in them are expensive.

For a large plant, the solids contact zone should be subdivided to limit the depth, and if necessary, a study should be made to determine the most economical combination of depth and number of cells.

A good arrangement is to place the cells in pairs, with an internal concentrator placed between each pair. This permits an economical and convenient arrangement of the flow lines for water, chemicals and sludge. For the Bacchus Marsh plant, two cells have been built initially.

Upflow Velocity.—The nominal upflow velocity at the top of the slurry pool (V) must be restricted to ensure satisfactory separation of the water from the solids.

A limited series of tests at about 15°C with Bacchus Marsh water[2] indicated that a safe nominal upflow velocity at the top of the pool would be 0.5 V_p, where V_p is the settling velocity of the individual floc particles, which is different for each kind of floc, and vaires with temperature.

The design must provide for the lightest kind of floc that will be produced, at the lowest operating water temperature, subject to the following considerations.

It has been shown[2] that V_p decreases with a fall in temperature, but that the velocity ratio V/V_p is independent of temperature. Hence, the nominal upflow velocity V may be altered, by varying the throughput as a routine part of plant operation, to suit the seasonal temperature. If this routine is to be adopted, then the fact that the demand is lower when the water is cooler allows the design to be based on summer conditions of demand and temperature.

It appears[2] that as the floc in an alum slurry accumulates turbidity, the particles become heavier and V_p increases. Advantage of this may be taken

TABLE 2

Kind of Floc	V_p at 15°C
Fragile floc, color removal with alum	0.12 to 0.24
Medium floc, algae removal with alum	0.20 to 0.30
Strong floc, turbidity removal with alum	0.24 to 0.35
Strong floc, lime-soda softening	0.24 to 0.35
Calcium carbonate granules	0.40 to 0.66

by starting up a new slurry pool with a reduced throughput (and upflow velocity) and gradually increasing to the design value.

Lime floc, on the other hand, appears to become lighter with age, but this is not likely to prove a limiting factor because such large quantities of slurry are drawn off that the remainder is always relatively fresh.

American manufacturers of upflow basins claim an output of 1 United States gal per sq ft per min from the top of the pool. This is equivalent to a nominal upflow velocity of 0.13 fpm. The kind of floc and the temperature are not stated.

Until sufficient published information is available, the settling velocity of the individual particles will need to be determined by test for the design of each proposed plant.

Table 2 gives the approximate range of values, in feet per minute, for various kinds of floc. (Various sources in the literature.) For the Bacchus Marsh plant, the following values, in feet per minute, were tentatively adopted.

	Fresh alum	Aged alum	Fresh lime	Aged lime
V_p at 15°C	0.27	0.35	0.37	0.30

As the plant will be required to handle both alum and lime floc, a conservative value of 0.28 fpm was adopted for V_p. The nominal upflow velocity V should not exceed 0.14 $(0.5\ V_p)$ fpm.

Separation of even the lightest floc likely to be encountered will be satisfactory[2] up to $V = 0.55\ V_P$, or for example, 0.15 fpm at 15°C; the operating temperature will exceed that in the summer.

If V is the nominal upflow velocity in feet per minute, and Q_2 is the output in gallons per minute,

$$V = \frac{Q_2}{6.23\ b^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

For the Bacchus Marsh design, for an output of 655 gpm and a two-cell unit,

$$V = \frac{328}{6.23 \times 26.0^2} = 0.078 \text{ fpm}$$

In practice, as will be reported, heavy floc is formed even with color removal with a much higher settling velocity V_P than assumed.

Slurry Concentration.—The concentration of the slurry at the top of the slurry pool is given[2] by

$$s = a\left(1 - \frac{V}{V_P}\right)^{1.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

in which s is the concentration of the suspension as measured by the wet solids fraction and V_P denotes the settling velocity of the individual particles of the suspension, in feet per minute, the adopted values of which were given previously.

TABLE 3

Value of s at 15°C for throughput of	671 gpm	1007 gpm
Fresh alum slurry	0.100	0.070
Aged alum slurry	0.115	0.091
Fresh lime slurry	0.187[a]	0.150
Aged lime slurry	0.169	0.125

[a] This value of s is above the range for which Eq. 9 holds but is sufficiently accurate and will be used.

A value of 0.17 for alum slurry and 0.27 for lime slurry[2] has been adopted for the parameter a, and these values have been confirmed in operating the completed plant.

The values calculated from Eq. 9, and using the appropriate value of a, V and V_P, are given in Table 3.

The Concentrator.—The zone of the concentrator is one of still water into which the slurry is diverted so that the solids may settle and concentrate to the desired degree before being removed from the plant. It should be located so as not to obstruct the upflow zone or interfere with the flow there. A photograph of the concentrator is shown in Fig. 4.

When, as in the present design, a slurry weir is used to control the level of the top of the slurry pool, the concentrator must be located outside the upflow zone and just below the slurry weir where it can receive the overflowing slurry.

The concentrator at Bacchus Marsh is located between the twin upflow zones, and part of one side wall of each of the latter forms opposite sides of the concentrator. Each wall common to the concentrator and an upflow zone is terminated at the proper height to form the slurry weir for that upflow zone.

The total length of slurry weir (L) and the area (A) of the concentrator is designed to accommodate the overflowing and settling slurry.

After trial by the methods to be described, the size of the concentrator was fixed at 16.75 ft square, so that its area is 280 sq feet and the length of the slurry weir is 33.5 ft.

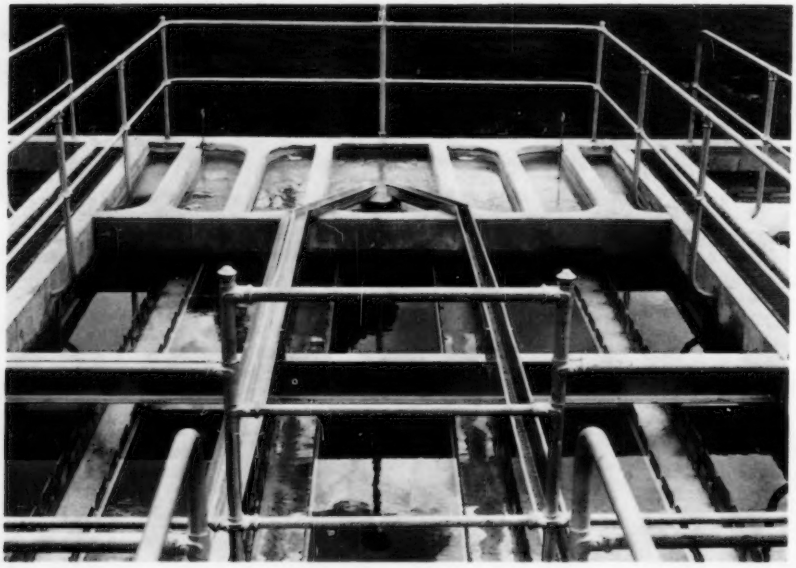

FIG. 4.—CONCENTRATOR, WITH MIXING BASIN IN BACKGROUND

Rate of Precipitation.—The rate of precipitation p, in pounds of wet solids at unit solids fraction, settling through one square foot in one minute, in the still water of the concentrator is given by[2]

$$p = w\ s\ V_p \left(1 - f\ s^{2/3}\right) \quad \ldots\ldots\ldots\ldots\ldots (10)$$

in which w is the weight in pounds per cubic foot of wet solids at unit solids fraction (64.5 lb per cu ft being the adopted value in this design); and f is a parameter.

It has been shown[2] that the value of f was 2.78 for both the alum and lime floc in settling experiments at Bacchus Marsh, but that this value was not consistent with the theoretical value of $f = a^{-2/3}$ when a of Eq. 9 was obtained from upflow experiments with the same flocs. The reason for this was considered

to be the difference in the shape of the particles formed under the two different sets of conditions.

The value of f for alum floc, corresponding to a = 0.17, is 3.89, and for lime floc, corresponding to a = 0.27, is 3.34, and these values of f will be used in calculating the rate of precipitation.

The rate of precipitation, p_1, when the solids are withdrawn from the bottom of a container or concentrator at the packed solids fraction, at the same rate as they precipitate is given[2] by,

$$p_1 = \frac{w\ s\ V_p \left(1 - f\ s^{2/3}\right)}{1 - \frac{s}{s_p}} \quad \ldots\ldots\ldots\ldots (11)$$

in which s_p denotes the packed solids fraction.

The values of p and p_1 at 15°C for the various kinds of slurry, at the concentrations overflowing the slurry weir, as given in Table 3, and calculated from Eqs. 10 and 11 are shown in Table 4.

Sludge Concentration.—The slurry settling in the concentrator will pack at the packed solids fraction, which in the case of the floc produced in the Bacchus Marsh experiments, is a parameter with a value of 0.3.

TABLE 4

Throughput	f	V/V_p	671 gpm			1007 gpm		
			s	p	p_1	s	p	p_1
Fresh alum slurry	3.89	0.27	0.100	0.29	0.44	0.070	0.31	0.32
Aged alum slurry	3.89	0.35	0.115	0.32	0.52	0.091	0.38	0.39
Fresh lime slurry	3.34	0.37	0.187	0.51	1.35	0.150	0.63	1.26
Aged lime slurry	3.34	0.30	0.169	0.48	1.10	0.125	0.55	0.94

After settling to the packed solids fraction, the Bacchus Marsh sludge will compact to within 2% or 3% of unit solids fraction in a few days. At the end of this period, the water still in the sludge is largely water of crystallization which must be dried off.

A great deal of caution must be exercised in estimating the value of the initial and packed solids fraction of other flocs, as shown by the following summary of American experience.

> "Some sludges will concentrate to 27 to 33 per cent wet solids, while others can only with difficulty be concentrated to ten per cent in the plant. Where withdrawal is intermittent, the concentration will vary quite considerably during the blow-off period."

> "Much less water will be wasted in concentrating and disposing of a dense crystalline calcium carbonate slurry than the bulky, flocculent slurries containing hydroxide floc, although the former appears much thinner than the latter."

Tests should be made, preferably in a working model, to determine the initial and packed concentrations of wet solids that may be expected and, hence,

the approximate amount that will have to be handled in the various parts of the plant.

Rate of Withdrawal.—The rate of flow over the slurry weir is $\frac{P}{w\,s}$ cu ft per min; the same as the rate of slurry removal.

In order to maintain the balance of solids in the concentrator, the rate of withdrawal of slurry at concentrator s_p must be $\frac{P}{w\,s_p}$ cu ft per min.

As will be seen later, the concentrator was designed on a liberal basis for the maximum rate of slurry production, when the raw water has a hardness of 550 ppm. The hardest water treated to date (1961) had a hardness of 256 ppm with a correspondingly lower rate of slurry production. It has been possible to retain the slurry in the concentrator until it compacted nearly to unit solids fraction and the wet solids are withdrawn at a concentration of s = 0.85 to 0.95. Thus, the rate of withdrawal averages $\frac{P}{0.9\,w}$ cfm. Experience has indicated that even at the maximum rate of slurry production, it may be possible, with a concentrator of this size, to retain the slurry in it until it compacts nearly to unit solids fraction.

Upflow Velocity Over the Concentrator.—The rate of displacement upwards of the water separating out from the settling solids is

$$\frac{P}{w\,s} - \frac{P}{w\,s_p} = \frac{P}{w}\left(\frac{s_p - s}{s_p\,s}\right) \text{ cfm} \quad \ldots\ldots\ldots\ldots (12)$$

Let A be the theoretical top area of the concentrator in square feet. Then the average upwards velocity of the displaced water at the top of the concentor is

$$\frac{P}{w\,A}\left(\frac{s_p - s}{s_p\,s}\right) = \frac{p_1}{w}\left(\frac{s_p - s}{s_p\,s}\right) \text{ fpm} \quad \ldots\ldots\ldots\ldots (13)$$

Substituting in this expression the value of p_1 from Eq. 11, it is found that the average upward velocity of the water rising out of the concentrator reduces to V the same as the nominal upflow velocity in the upflow zone at the level of the slurry weir, for

$$\frac{p_1}{w}\left(\frac{s_p - s}{s_p\,s}\right) = \frac{w\,s\,V_p\left(1 - f\,s^{2/3}\right)s_p}{s_p - s}\;\frac{s_p - s}{w\,s_p\,s} \quad \ldots\ldots (14)$$

$$= V_p\left(1 - f\,s^{2/3}\right)$$

$$= V_s = V$$

(It is obvious that the settling velocity, V_s, of a settling suspension is the same as the nominal upflow velocity, V, required to hold the same suspension at rest.)

The Slurry Weir.—The function of the slurry weir is to control the top of the slurry pool by discharging the surplus slurry into the concentrator, thus ensuring that slurry is not decanted. As the concentration in the slurry pool is constant at a given throughput, the volume of slurry and, hence, the level of its top can be kept constant by withdrawing old solids at the same rate as new ones are formed. It is obviously desirable that they should be withdrawn only after solids contact is complete. They should be withdrawn at a location where concentration of slurry is already high and at a location where there is no interference with the upflow.

It was conceived that these requirements might be met simply by causing the new solids entering the bottom of the pool to displace the old at the top and

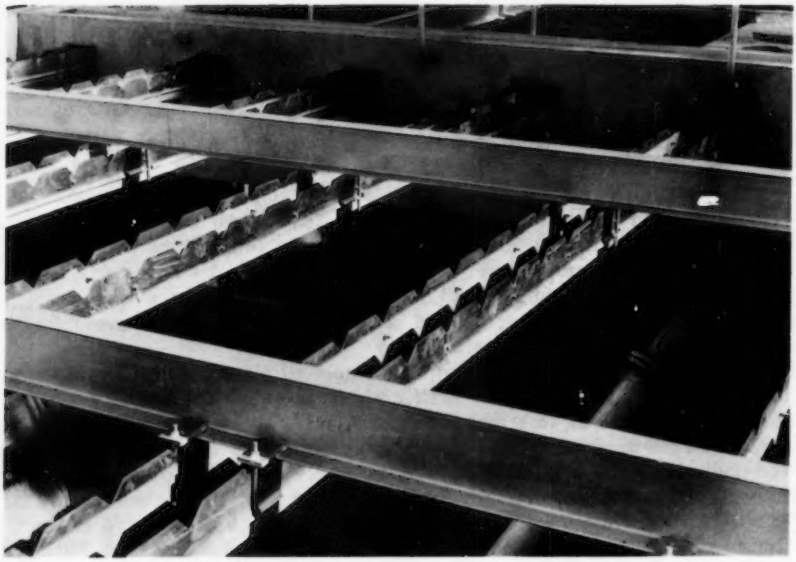

FIG. 5.—SLURRY WEIR SEPARATING UPFLOW BASIN AND CONCENTRATOR BEYOND

allowing the surplus slurry to overflow a weir at the appropriate level below the water surface into a concentrator placed to receive it. Fig. 5 is a photograph looking down through one of the upflow basins and shows the slurry weir across the center, with the concentrator, the dark area, beyond it.

The idea of a slurry weir was tested in a model upflow basin throughout a long series of experiments. It was consistently successful and was incorporated in the Bacchus Marsh plant. By its use, simple and certain control of the top of the slurry pool and the other requirements enumerated previously have been attained, and the old solids overflow the weir in a continuous stream at the same rate as the new ones are produced, thus automatically removing

them from the slurry pool at the correct rate, no matter how that rate may vary from time to time.

A decrease in the rate of throughput will reduce the upflow velocity and cause the top of the slurry pool to fall to a lower level, with the solids at a higher concentration. Within reasonable limits, this in itself is unlikely to have serious consequences, as the time of solids contact will not greatly alter. In a plant using a slurry weir, no solids will reach the concentrator until sufficient new solids have built up the slurry pool to again overflow the weir. When this occurs, an increase in the flow through rate will cause the slurry to spill over the weir faster than the design rate.

If the concentrator is adequate in size, and the rate of withdrawal is set at or above the average rate, it will cope with the higher rate of flow of slurry with only a slight temporary rise in slurry pool level, and no floc will pass off with the decanted water. In plants not provided with a slurry weir, the carry over of floc under such conditions may assume serious proportions.

Provision has been made to adjust the height of the weir by means of movable boards, so that the top of the slurry may be brought as close to the decanting level as safe operation will permit.

This will increase the time of solids contact for a given throughput, or conversely, permit an increase of throughput for a given time of contact.

Flow of Slurry Over the Weir.—The slurry in the pool, under the water, will overflow a weir in a similar way to water in air. When the Bacchus Marsh plant was being designed, no information was available as to the probable horizontal velocity of the slurry over the weir, but because its density is so near that of water, it was thought it would be very low, perhaps in the same order of magnitude as the upflow velocity, and a liberal allowance was made for the possible head of slurry on the weir.

However, it has been found that in the completed plant the horizontal velocity is about 3 fpm, under certain artificial conditions. The velocity was calculated from the rate of flow and the cross-sectional area of the overflowing slurry, attempts to obtain a direct measurement of the velocity having so far proved unsuccessful. In order to obtain an easily measurable depth of slurry on the weir, slurry was pumped from the concentrator through one upflow basin, at a rate equivalent to 5.4 cfm at s = 1.0, which is nearly five times the maximum rate of slurry production.

The rate of flow of slurry over the weir (length L, in feet) is $\frac{P}{w\ s}$ cfm, and its depth (or head on the weir) is h, in feet.

Then

$$L\ h\ v = \frac{P}{w\ s} \quad \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

If v is assumed to have the preceding value, 3 fpm, the values of L and h may be selected. It is suggested that for plain clarification plants using coagulation only, L need only be about one fourth or one fifth of the length of one side for each basin. For lime-soda softening of the Bacchus Marsh water the total length L of 33.5 ft is ample, as the depth of slurry on the weir during maximum production will probably not exceed 1 in. At an output of 983 gpm and extreme conditions of hardness (550 ppm) P = 75 lb per min and s = 0.135.

Assuming v = 3 fpm, then

$$h = \frac{P}{w\ s\ L\ v} \quad \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

$$\frac{75}{64.5 \times 0.135 \times 33.5 \times 3.0} = 0.085 \text{ ft}$$

The test just described showed that a value of h as high as 5 in. could be used without detrimental effects, and any value up to this amount could be selected to suit the required size of the concentrator.

Area of Concentrator.—The top area of the concentrator is obtained by dividing the rate of slurry removal (P, in pounds per minute of wet solids at s = 1.0) by the rate of precipitation (p or p_1 pounds per square foot per minute of wet solids at s = 1.0).

It is necessary to allow sufficient area to settle the slurry from the maximum throughput (1007 gpm) the plant can reasonably be expected to handle, at the maximum hardness (550 ppm) likely to be encountered.

This amounts to (P =) 75 lb per min of wet settled solids of lime slurry at units solids fraction. (The rate of alum slurry removal is so low that it need not be considered.)

The slurry will be a mixture of fresh and aged, and the probable rate of precipitation will be about 1.1 psf per min during withdrawal. (See Table 4.)

It may be necessary to operate for part of the time without withdrawing slurry from the concentrator, in which case the rate of precipitation would fall to about 0.6 psf per min. Making allowance for this, a rate of precipitation of 0.8 psf per min will be assumed.

Then

$$A = \frac{P}{p_1} = \frac{75}{0.8} = 93.5 \text{ sq ft}$$

The values of the quantities used in the preceding calculations are tentative and require further experimental verification, and it was thought that the flow of the slurry over the weir might cause eddying in the concentrator and, thus, depart from the ideal case. (This has not occurred in practice.) It was, therefore, considered that the actual area of the concentrator should be several times the theoretical value, and an area of 280 sq ft was adopted.

This area is three times that theoretically required for the maximum rate of precipitation, corresponding to a throughput of 1007 gpm and a hardness of 550 ppm. Experience has shown that the plant can be operated at an even higher rate than this and that the concentrator area is not unduly large.

Volume of Concentrator.—The solids in a suspension of floc settle quite rapidly to the packed solids fraction, the value of which can be readily determined. It will approximate 0.3 for these flocs. Thereafter, compaction is slow, several days being required for the solids to compact to within 2% or 3% of unit solids fraction.

Due to the high cost of a concentrator built into the actual structure of the treatment plant, it should be designed only to concentrate the solids to the packed point. This is the best compromise between economy of structure and waste of "blow-off" water. The solids should then be withdrawn and the rest of the compaction completed in cheaper external chambers or lagoons.

If the rate of withdrawal of solids were the same as the rate of entry to the concentrator, it need have only sufficient room to allow the solids to settle. Allowance must be made, however, for errors in setting the withdrawal rate.

The best shape for the concentrator is considered to be an inverted pyramid. This shape causes the slurry to settle to a small area whence it can easily be withdrawn, through a pipe, under hydrostatic pressure. This shape will conveniently fit below the slurry weir, the length of which more or less determines the size of the top of the concentrator. The walls must have a slope of at least 1.65 vertical to 1.0 horizontal to permit the solids to roll freely down them.

The volume of the concentrator, the size of which is fixed by these considerations and its surface area, should then be checked to see whether sufficient storage space is available to allow for errors in settling the withdrawal rate.

In the design used as an example, the maximum rate of slurry withdrawal is $\frac{P}{w\, s_p} = 3.88$ cfm, and the volume of the concentrator is 1280 cu ft.

For reasons already given, the top area of the concentrator was fixed on a liberal basis, and that led to a very large storage volume permitting considerable error in setting the withdrawal rate.

Sludge Disposal.—It was assumed that sludge from the concentrator at a concentration of s = 0.3, and filter wash waste water at a concentration of s = 0.05 would be pumped to the drying beds and there allowed to settle and compact, the supernatant liquor being returned to the plant. For convenience in pumping, the wastes are first discharged under gravity into a waste water tank before being pumped to the drying beds. It is worth noting at this point that because of the high concentration (and low volume) of the slurry withdrawn from the concentrator at ordinary rates of slurry production, it and the filter washwater can be stored, settled and compacted in the waste-water tank, and the supernatant liquor pumped back from there to the filters and the compacted sludge, with a minimal amount of water, pumped to the drying beds. More water is recovered by doing this than if the whole bulk is pumped to the drying beds.

Decanting.—The water leaving the top of the slurry pool in the sedimentation basins must be drawn off without causing currents or eddies that will disturb the slurry.

The ideal method of decanting would preserve the vertical stream-lined flow, uniformly distributed over the entire surface in which the water rises from the slurry pool.

Calculations based on published information[3] showed that peripheral decanting would require a great depth of clear water above the top of the slurry even to keep the maximum upflow velocity at the level within 30% of the average. As a result, a system of collecting troughs drawing off at a number of points uniformly disposed over the surface of the water was installed.

Comparative estimates indicated that the most economical arrangement in this case was to divide the top water area of each upflow basin into six equal rectangular spaces, 4 ft 4 in. wide, with a collecting trough 9 in. wide along the center line of each.

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[3] "Settling Characteristics of Suspensions in Water Treatment Processes," by A. A. Kalinski, Journal, AWWA, Vol. 40, No. 113, February, 1948.

Both sides of each trough have V-notch weirs at intervals of 11 in., and are shown operating in Fig. 6. The slurry weirs were fixed 6 ft below the decanting level, and with the decanting arrangements previously outlined, the top of the slurry is perfectly level.

In order to facilitate exact leveling, the V-notch weirs were cut in a number of separate stainless steel plates attached to the sides of the decanting troughs with bolts through slotted holes. It has been stated[4] that submerged orifices provide uniform effluent collection without the need for the accurate levelling required for weirs. This is correct within limits depending on the relative size of the orifices and the collecting trough or pipe. However, orifices are less accessible for inspection and cleaning than weirs.

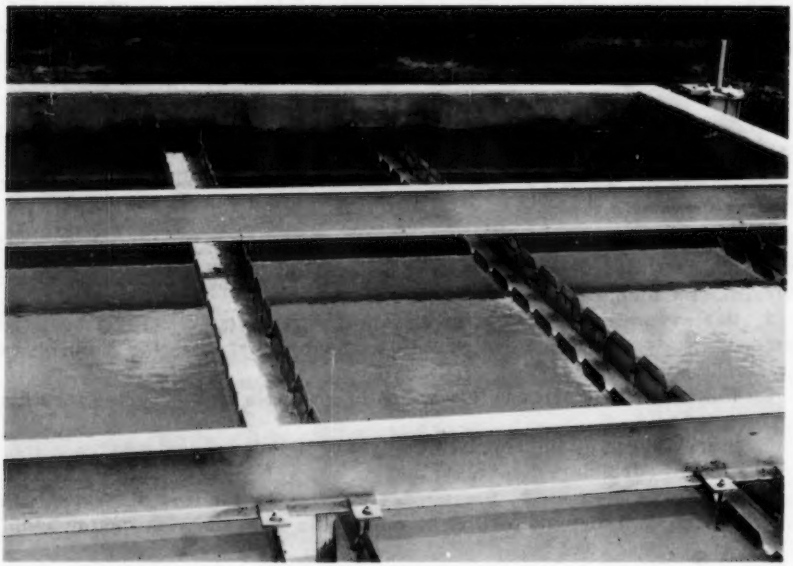

FIG. 6.—COLLECTING TROUGHS

Decanting Over the Concentrator.—As the slurry settles and compacts in the concentrator, water separates out from it. To prevent the water from flowing back over the slurry weirs and disturbing the incoming slurry, it should be decanted over the concentrator.

The upflow velocity of the decanted water above the concentrator equals the nominal upflow velocity above the slurry pool when the top area of the concentrator is designed to suit and is receiving the maximum rate of slurry production. Under such conditions, the same decanting arrangements as used over the slurry pool would be required over the concentrator and were provided.

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4 Journal, AWWA, April, 1951, p. 278.

In this design, however, the concentrator area is greater than that theoretically required, and this reduces the upflow velocity but not the rate of decanting. At rates of slurry production less than the maximum, the rate of decanting itself will be proportionately less, and provision was made to allow it to be varied to suit the rate of slurry production.

Additional sets of V-notch plates were installed, each of which slides horizontally along the face of one of the fixed V-notch plates over the concentrator. Composite V-notches were thus formed. The level of their bottoms can all be varied by the same amount, by sliding the moveable set.

Retention Time.—The time of solids contact is the true criterion of basin performance. However, the retention time is a useful measure of the size and cost of the basin to many engineers, although as T. R. Camp has noted,[5] in connection with horizontal flow basins, it has no real significance.

The total volume of the mixing basin and the two upflow zones is 18,400 cu ft. Hence, the retention time for a throughput of 671 gal per min is about 170 min and for a throughput of 1007 gal per min, about 114 min.

Judging by the claims of reputable American manufacturers of upflow basins these times are conservative, but that was the intention.

Operation.—The plant of which the upflow basins described in this paper form a part is operated by one man working one shift. This has been achieved in respect to the basins by the availability or provision of the following principal features:

1. A raw water storage large enough to smooth out fluctuations in quality, which are not large or sudden anyway, so avoiding setting of the chemical feed rates more than once a day;
2. A clear water storage large enough to balance out hourly fluctuations in demand, thus permitting a constant throughput and eliminating disturbance of the slurry pool;
3. Uniformly expanding upflow zones of a simple type ensuring stability of the slurry pool;
4. Slurry weirs giving complete control of the slurry pool and the automatic removal of surplus precipitates; and
5. Automatic shut down of the plant by stopping the flow of raw water and slurry withdrawal and the feeding of chemicals when the clear water basin is full.

The operator starts the operation manually, fills chemical hoppers, makes necessary tests, adjusts feed rates, attends to maintenance and cleaning duties, and then leaves the plant unattended to complete its day's run.

RESULTS

Softening.—The maximum total hardness encountered in the raw water from the time the plant commenced operation in August, 1959 to 1961, it was 256 ppm. Non-carbonate hardness varies from 45% to 55% of the total hardness. Magnesium hardness varies from 65% to 75% of the total hardness.

No difficulty has been experienced in reducing the total hardness of the settled water leaving the sedimentation basins to the desired value—60 ppm with

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[5] "Sedimentation and the Design of Settling Tanks," by T. R. Camp, Transactions, ASCE, Vol. 111, 1946, p. 895.

doses of lime and soda as calculated previously, reduced slightly as necessary to allow for the sodium hydroxide in the coagulant.

With such a high proportion of magnesium hardness, sodium aluminate has been found as usual to be the only satisfactory coagulant for softening. A dose of 15 ppm at a water temperature of 73°F is sufficient to prevent floc haze in the clear water zone, thus almost eliminating the load on the filters; but when the water temperature falls to 50°F, the dose has to be increased to 30 ppm. With sufficient excess lime and soda-ash, the total hardness has been reduced in warm weather, as an experiment, to 34 ppm.

Hydroxide alkalinity is usually carried at 6 ppm to 12 ppm in the settled water in warm weather, but with the higher cold weather aluminate dosing, may rise to 40 ppm or 50 ppm. The pH of the raw water is usually about 8.0 and that of the (lime-softened) settled water 10.0. The settled water is practically in chemical balance to calcium carbonate, being usually slightly supersaturated. The settled water is dosed with 0.5 ppm and the filtered water with 1.0 ppm of sodium hexametaphosphate and no corrosion or incrustation has yet been found or reported.

The chlorine demand of the raw water is variable, but high, whereas that of the settled water is 0.5 ppm. The maximum raw water turbidity encountered was 450 ppm while softening. The settled water leaves the sedimentation basins with a turbidity of 1.0 ppm to 1.5 ppm, due mostly to minute particles of floc. The filters reduce the turbidity to less than 0.3 ppm, and are, thus, useful principally as the last line of defense.

The maximum color in the raw water was over 140 ppm. The settled water leaving the basins has a color of 4 ppm to 5 ppm, and the filtered water a color of less than 3 ppm.

Tastes and odors occur intermittently without warning, and have compelled regular dosing of the settled water with activated carbon to eliminate them.

Under the conditions so far encountered, the plant will operate satisfactorily at a water temperature of 49°F and give settled water of the quality previously described, at a throughput of 1900 gpm—2.83 times the design rate, the highest available with the present hydraulic conditions of the raw water supply. That is, with raw water of the previously stated quality and temperature and this particular process, the time of solids contact has been reduced to 32 min, the retention time to 60 min, and the upflow velocity (V) at the top of the slurry pool has increased to 0.22 fpm.

Clarification.—In the autumn and early winter (August to December) of 1960, the hardness of the raw water fell below 70 ppm, and the chemical process was changed from lime-soda softening to plain clarification, using alum and activated silica as coagulants.

The maximum rate tested was 1800 gpm (2.68 times the design rate) corresponding to a time of solids contact of 34 min, a retention time of 63 min, and an upflow velocity at the top of the slurry pool of 0.21 fpm. The temperature of the water was 65° to 75°F the settled water color 7 ppm and its floc turbidity 2.5 ppm.

Filtration.—Because the previously noted hardness, turbidity, and color are the maxima at the stated rates and temperatures, it is the writer's present opinion that if considerations of economy require it and if quality permit it, filtration of the effluent from the separation basins could be dispensed with.

Experimental operation for a few months without filtration at Bacchus Marsh resulted in slight after precipitation of floc near the ends of some mains. This precipitation was easily removed by flushing.

This is not intended to mean that filters should be dispensed with in any case in which a community can afford them. Because they are a defense against operational errors, they provide contact time for chlorination and they improve the quality of the finished product.

CONCLUSIONS

A simple economical upflow solids contact separation basin can be scientifically designed from first principles.

The only basin information required is a knowledge of the characteristics of the floc which will be formed with the water and treatment process to be used. These can easily and cheaply be obtained from model tests.

The basin has no waste space, no mechanical equipment, has a high performance and can be used for softening, or clarification, or both.

The inclusion of a simple structural feature—a slurry weir—will give complete, positive, and automatic control of the slurry pool.

ADDITIONAL REFERENCE

1. "Lime and Lime-Soda Sludge Disposal," by A. P. Black, Journal, AWWA, Committee Report, September, 1949, p. 819.

APPENDIX.—NOTATION

Letter symbols adopted for use in this paper are listed here for ease of reference and for the convenience of discussers:

A = The top area of a concentrator (square feet);

f,a = constants depending on the shape of the particles;

h = the depth of slurry overflowing the weir (feet);

L = the total length of slurry weir (feet);

P = the rate of removal of wet settled solids at unit solids fraction from the slurry pool (pounds per minute);

p = the rate of precipitation of wet solids expressed as the weight of unit solids fraction falling through unit horizontal area in unit time (pounds per square foot per minute);

p_1 = the rate of withdrawal from a concentrator of wet solids at the packed solids fraction, expressed as the weight of wet solids at unit solids fraction per unit horizontal area at the top of the concentrator in unit time (pounds per square foot per minute);

Q_1 = the throughput, that is, the total rate of flow of water and slurry through the solids contact zone (Imperial gallons per minute);

Q_2 = the output, the net rate of flow of water leaving the solids contact zone (Imperial gallons per minute);

s = the concentration of a suspension, as measured by the wet solids fraction;

sp = the packed solids fraction;

t = the time of solids contact in minutes;

V = the nominal upflow velocity in an upflow zone in which no particles are present, relative to the same frame (feet per minute);

V_S = the settling velocity of a suspension relative to an external frame (feet per minute);

V_P = the settling velocity of the individual particles of the suspension, relative to the same frame (feet per minute);

v = the horizontal velocity of the slurry overflowing the weir (feet per minute);

w = the weight of wet solids per unit volume of a suspension of unit solids fraction (pounds per minute); and

z = the volume of a solids contact zone (cubic feet).

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SANITARY ENGINEERING DIVISION
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RESEARCH ON COMPOSTING OF CITY REFUSE AND NIGHTSOIL

By Yonosuke Kaibuchi[1]

SYNOPSIS

Several years of research, with both a pilot plant and a 25-ton per day proto-type plant, were conducted with financial and technical assistance from the World Health Organization. Studies included evaluation of two types of high-rate mechanical digesters and forced air curing bins. Equipment is described, processes evaluated, and detailed chemical and bacteriological results given.

INTRODUCTION

In the interest of upgrading the overall sanitation environment of Japan, many Japanese cities urgently require a sanitary and economical method of disposal for organic wastes such as municipal refuse and nightsoil.

The continuous use of chemical fertilizers alone, is causing a decrease in soil quality in many agricultural areas and a large quantity of organic matter such as manure is required to supplement these soils. A disposal process which treats human excreta and refuse in a sanitary manner and at the same time manufactures plant nutrients in an easily usable form, is aerobic composting.

Previously, the treatment of organic wastes in Japan has been accomplished by means of anerobic or semi-anerobic digestion. For instance, in

Note.—Discussion open until April 1, 1962. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SA 6, November, 1961.

[1] Civ. and San. Engr., Dir., Sewerage Dept., Constr. Bur., Kobe City, Japan.

1938 Kobe City constructed 54 composting cells to be used as a demonstration plant for the disposal and utilization of organic wastes. The cells were constructed on the site of an existing incineration plant and were operated on a batch basis. The digestion time utilized was about 17 days or 21 cycles per yr. Each cell was loaded with about 2.63 metric tons of raw material at the start of each cycle giving a yearly disposal capacity of 55.1 metric tons. Finished compost amounted to 34.7 metric tons per yr per cell and was welcomed by people in the area as fertilizer for their vegetable gardens. However, these composting cells failed to develop to the extent that they replaced existing incineration plants as a method of disposal of city waste. The main reasons for this were that the cells required much hand labor, long detention periods and a comparitively large amount of land, which is not available in the cities of Japan. If a mechanized plant that does not require large areas and that does not develop objectionable odors could be developed, composting would certainly be welcomed and would take the place of the conventional incineration plant. This type of plant would also play an important role in supplying organic fertilizers for the farming areas of Japan.

In October, 1955, the Government of Japan and the World Health Organization (WHO) entered into an agreement for a research study referred to as the "Environmental Sanitation Project," namely, a study of high-rate composting of night soil and refuse. The proposed plan of action was to make preliminary studies to determine sanitary, efficient, and economical methods of high-rate composting. On the basis of the preliminary data, pilot plant studies were to be made and from this information, a proto-type plant for the composting of human excreta and other organic waste materials was to be constructed and operated. The project proposal to the Japanese Government was initiated in 1954 by Paul Bierstein, M. ASCE, regional advisor on environmental sanitation, WHO Western Pacific Office, Manila. It was obvious that the proposed plan of action would best be conducted in a city and it was subsequently arranged that Kobe City would be the site of the studies.

Prior to formal conclusion of the project agreement, a small scale pilot plant was built. Construction started in December, 1954, and trial operation began in March, 1955. After overcoming initial operational trouble, experiments were conducted with the pilot plant from May, 1955, to July, 1956. From these data and from suggestions by J. R. Snell, F. ASCE, who acted as consultant for WHO, a proto-type plant was constructed. The results of the pilot plant and proto-type plant studies are detailed in reports I through VI previously submitted to WHO. These reports have been combined and summarized herein.

"High-Rate Composting" may be described as a process in which putrescible materials in organic wastes such as carbohydrates and proteins are oxidized and stabilized biologically in a matter of days, while the pathogenic organisms in these wastes are destroyed and made harmless by the heat generated in the process.

The basic features of high-rate composting have been established by the experimental programs previously conducted at Michigan State University, East Lansing, Mich., and other institutions. The process is outlined as follows.

Raw waste materials are first ground and are fed continuously to a mechanical digester for stabilization. Material in the digester is stirred slowly either continuously or at short intervals. Digested material is recycled and mixed

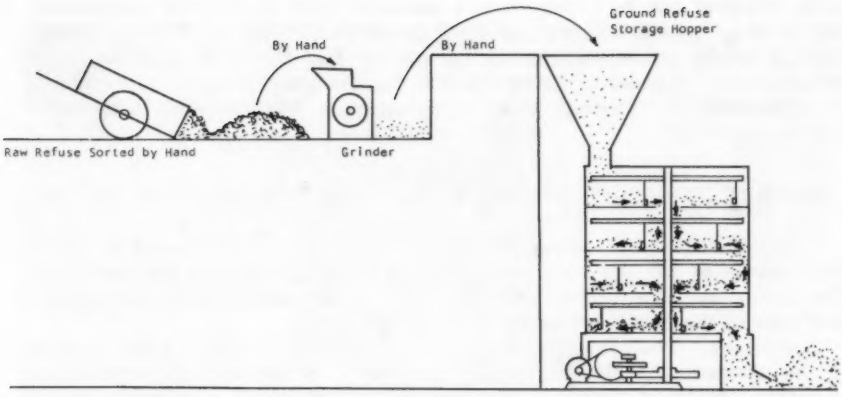


FIG. 1.—FLOW DIAGRAM OF HIGH RATE COMPOSTING PILOT PLANT KOBE CITY, JAPAN

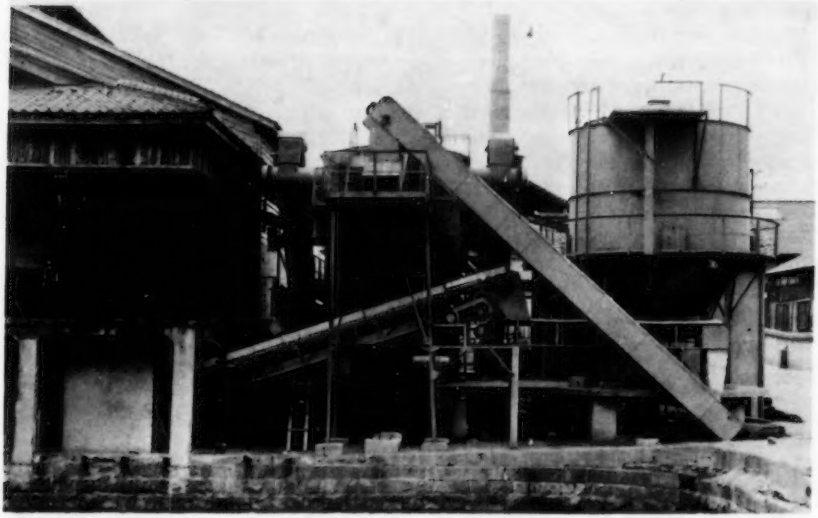


FIG. 2.—THE STORAGE BIN AND DIGESTER

with the raw ground waste and the entire mass is kept in an aerobic and thermophilic condition. With the use of such a process and under favorable conditions, compost can be produced in a matter of days. In windrow composting, that is, when ground material is piled on the ground and periodically turned, about 3 weeks are necessary for the material to obtain the same degree of decompositon. In order to distinguish it from the conventional windrow methods of composting, the process under examination will be referred to as "high-rate composting."

EXPERIMENTS WITH THE "NAKAMURA" SMALL-SCALE PILOT PLANT

A small-scale pilot plant for high-rate composting was erected to collect data necessary for the construction of the larger, 25-ton per day type plant. The pilot plant was called "Nakamura Plant" because it was intalled on the premises of the Nakamura Incineration Plant.

Equipment.—The refuse to be composted was sorted by hand on the platform of the incineration plant in order to remove the metalic and porcelainous elements contained therein. The material was then put into the grinder by hand. The ground material was put into the storage hopper by hand and then was fed continuously to the digester (See Figs. 1 and 2).

Description of Equipment.—

Grinder (Figs. 3 and 4).—A "Swing hammer type" grinder with a capacity of 0.5 tons per hr was adopted. This grinder was connected to an electric motor of 7.5 hp and 1,800 rpm. The original clearance of the hammer discs was 12 mm, center to center; however, because of persistent clogging which impeded the operation, the clearance was widened to 25 mm while the undersscreen was removed.

The high percentage of ashes and dirt in the refuse wore the hammer edges to a remarkable degree. Therefore, many means of enhancing the hardness of hammer knives, were attempted.

Storage Hopper.—A hopper with a storage capacity of 1 cu m was fitted, to the digester for continuous feeding (Fig. 2). Due to the bridging action of shredded garbage at the discharge port, the desired continuous discharge was not realized. An Archimedis screw and an electric vibrator were used to eliminate this trouble but without success. The same trouble was experienced with the storage bin of the proto-type plant that was built afterward. As long as the hopper was cone-shaped, trouble was experienced with bridging regardless of the size of the outlet.

Digester (Fig. 5).—Details of this digester, (Fig. 6) were as follows: Diameter 7.5, ft height 7.8 ft, 4-stage continuous flow modified Earp-Thomas type, disposal capacity 0.6 tons per day (1.5 cu m per day). With a depth of material on each stage of 30 cm, the maximum detention time is calculated to be 3 days. Four revolving arms were connected to the revolving center shaft on each stage, and two plows were attached to each arm (Fig. 7). Afterward one of these plows was removed because composting material kept arching between the plows. The surface of the shell was insulated with rock wool 2-in. thick.

Reduction gears were modified to reduce the number of revolutions per min from 1/4 to 1/12, thereby increasing detention times from one day to three days.

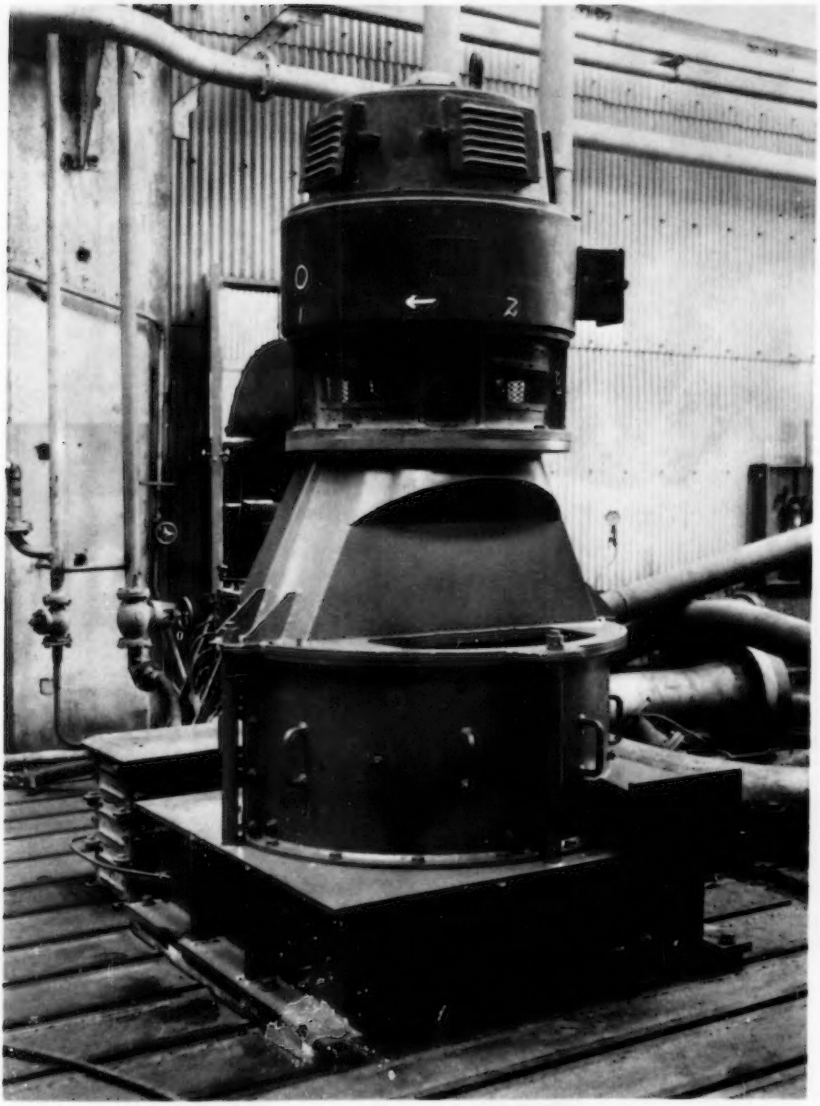


FIG. 3.—THE GRINDER

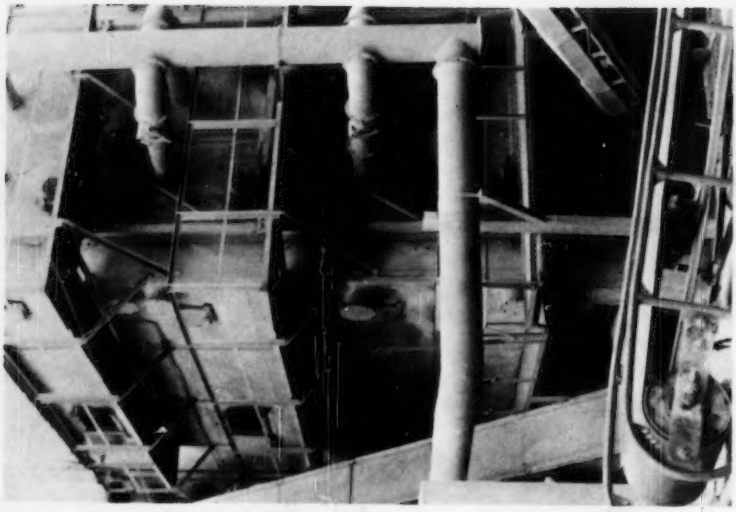


FIG. 5.—THE DIGESTER

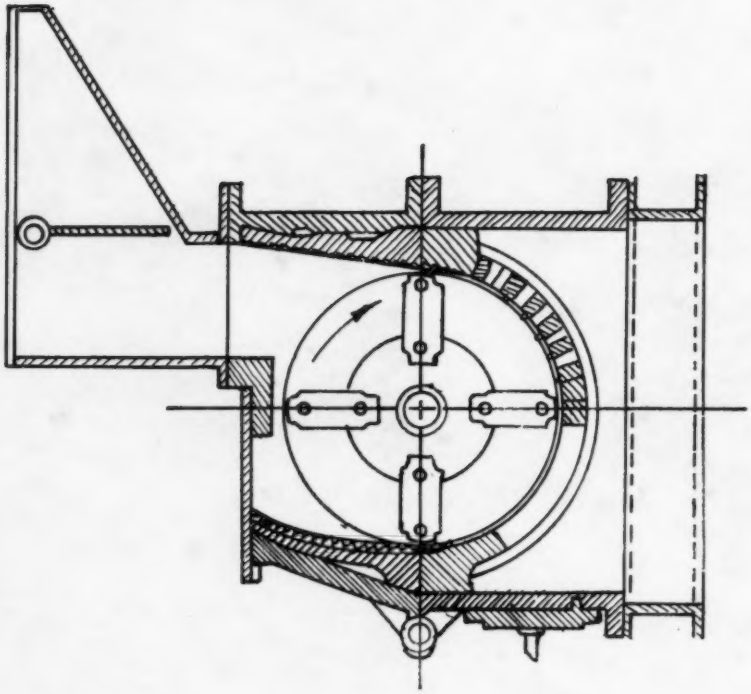


FIG. 4.—GRINDER DETAILS

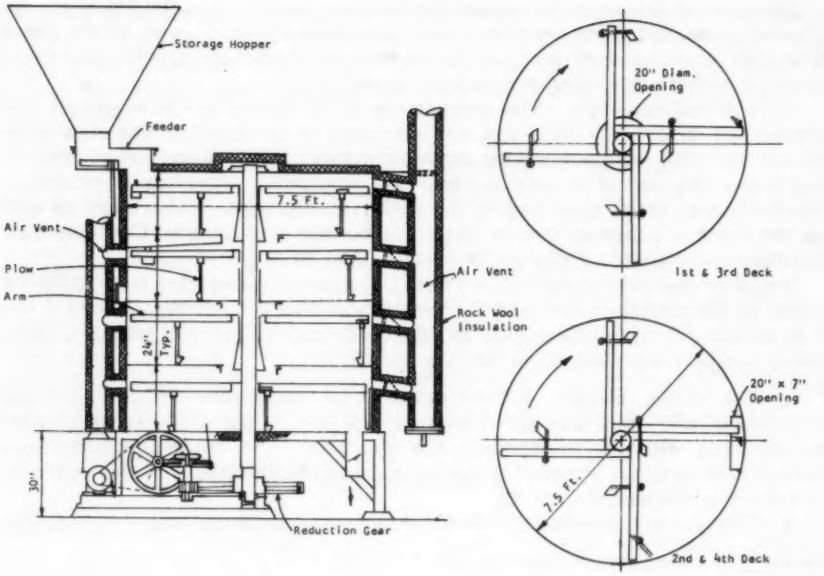


FIG. 6.—MODIFIED EARP-THOMAS TYPE DIGESTER

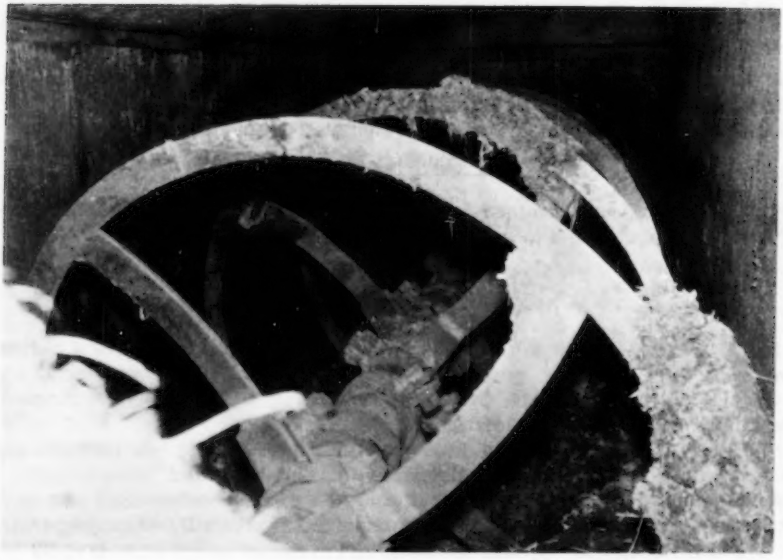


FIG. 7.—THE INTERIOR OF THE DIGESTER SCREW CONVEYER

Because of the large stresses and torques developed in the digester, the original arms and plows were replaced with much stronger ones. At the same time, the connection of the plow to the arm and that of the arm to the center shaft were thoroughly fortified to prevent slipping.

Operational Schedule.—The composting of ground refuse in windrows was started on March 1, 1955, and the operation of the digester was started on March 13, 1955. In operating the digester various troubles were encountered, and it was impossible to carry on high-rate composting experiments smoothly. Furthermore, much time had to be spent remodeling the laboratory as well as obtaining a complete set of chemical testing apparatuses. Consequently, the chemical analysis could not be started until May.

Sampling and Analyzing.—Five 1 kg to 2 kg samples were taken from different parts of the materials and mixed. From this mixture, grab samples of 1 kg-2 kg each were taken and uniformly mixed. From this uniform mixture, three 200-g samples were taken out for determination.

1. The 200-g sample (a) was dried at the temperature of 100°C. in the oven for 5 hr. Then, this dried sample was left in the air until it became equilibrated with the moisture in the air. Therefore, the weight difference between the original sample (a) and the sample equilibrated with the moisture in the air is the weight loss (b).

2. The sample equilibrated with the moisture in the air was crushed. The percentage of the weight loss is: $\frac{b}{a} \times 100 = A\%$

3. 2 g of the crushed material (c) was dried at the temperature of 110°C. in the oven for 5 hr. The percentage of the weight loss is: $\frac{d}{c} \times 100 = B\%$ (d is the weight of the moisture that evaporated). Therefore, the real weight loss, namely, the moisture content of the original sample is: $A + \frac{(100-A) \times B}{100}$

4. 10 g of crushed material mentioned in item 2 was ignited in a muffle furnace and the percentage of ash was calculated.

5. The percentage of carbon was calculated from

$$\text{Percentage of carbon} = \frac{100 - \text{percentage of ash}}{1.8} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

6. Nirogren was determined by the Kjeldahl method on 5 g of sample material with HCl being added to the material before drying.

EXPERIMENTS

Raw Material.—In an effort to obtain raw material with as low a carbon nitrogen ratio (c/n) as possible, refuse from only the commercial and residential districts of Kobe City were used. Table 1 indicates the range of constituents found in this material.

High-rate Composting.—The following information describes methods and results of experiments with the pilot plant high-rate digester.

Experiment 4 (starting up operation).—Two parts refuse and one part end product from the digester were mixed and fed at a rate 0.065 cu yd per hr. The depth of material on each deck during this experiment was about 20 cm and detention time in the digester was about 70 hr. Air vents were closed throughout this experiment. The temperature of the material varied as follows:

56°C top deck, 44°C second deck, 47°C third deck and 41°C bottom deck. Mean ambiant temperature was 20°C. When the end product from the digester was piled in windrows the temperature rose to from 60°C to 70°C and remained at this level for a week. Table 2 shows the average chemical analysis for this experiment.

TABLE 1.—RANGE OF CONSTITUENTS IN RAW MATERIAL[a,b]

Vegetable and putrescible	Paper	Soil	Straw	Metal	Glass and Porcelain	Wood	Textile
30-70	5-30	10-40	1-5	0.1-1	1-3	1-10	0.5-2

[a] Percentage by weight
[b] The unit weight of the raw material was 400-675 lb per cu yd and the initial moisture was 50% to 55%.

TABLE 2.—(Exp. 4)

Material	Moisture, percentage	Ash, percentage	Volatile matter, percentage	Carbon, percentage	Nitrogen, percentage	C/N, percentage
Feed material	57.7	46.9	53.1	29.5	1.24	23.6
End product from the digester	37.1	61.9	38.1	21.0	1.12	18.7
End product piled in windrows after 5 days	39.1	62.4	37.6	20.9	1.10	19.0

TABLE 3.—DEPTH OF MATERIAL AND TEMPERATURE (Exp. 5)[a]

		Top stage	2nd stage	3rd stage	4th stage
Depth of material, in centimeters	Maximum	30	28	25	30
	Mean	25.7	20.0	19.9	21.4
Temperature, in °C	Maximum	63	59	50	45
	Mean	58	47	45	39

[a] Mean ambient temperature: 27°C.

Experiment 5 (Effect of Non Seeding).—Experiment No. 5 lasted from June 22 to July 1, 1955. The conditions were: Continuous flow; 3 days detention and a feed rate of 0.065 - 0.131 cu yd per hr. The depth of material and temperature are indicated in Table 3 and chemical analysis in Table 4.

Because raw material varied during the three days, analyses were made at the beginning and the end of the experiment. From the variation of ash (shown

in Table 5), it was inferred that decomposition took place on the top and the 2nd stage.

Experiments 6 and 7 (Effect of artificially increased moisture content).—Experiments 6 and 7 were made with material, the moisture content of which had been artificially increased to 70%, 63%, and 60% by addition of water. The conditions were: Continuous flow; 3 days detention; the feed rate was 0.065 cu

TABLE 4.—CHEMICAL ANALYSIS (Exp. 5)

Material	Moisture, percentage	Ash, percentage	Volatile matter, percentage	Carbon, percentage	Nitrogen, percentage	C/N, percentage	P_2O_5, percentage	K_2O, percentage	pH
Raw Material A	50.2	49.7	50.3	27.9	0.92	30.3	-	-	6.2
Raw Material B	48.8	51.7	48.3	26.8	0.90	29.8	-	-	6.2
End Product	39.5	60.2	39.8	22.2	0.78	28.5	0.49	0.41	6.8

TABLE 5.—THE VARIATION OF MOISTURE AND ASH ON EACH STAGE (Exp. 5)

Variant	Material A	Material B	Top stage	2nd stage	3rd stage	4th stage	End Product
Moisture, percentage	50.2	48.8	47.2	46.1	42.9	40.2	39.5
Ash, percentage	49.7	51.7	58.0	59.6	58.9	61.0	60.2

TABLE 6.—(Exp. 6 and 7)[a]

		Top stage	2nd stage	3rd stage	4th stage
Raw Material of 70% moisture content, in °C	Maximum	42	43	40	39
	Mean	38.5	39.5	39.0	37.3
Raw Material of 63% moisture content, in °C	Maximum	44	41	40	39
	Mean	40.0	39.0	38.4	37.8

[a] Mean ambient temperature: 28°C.

yd per hr. The material was ground refuse, 2 parts; end product, 1 part. The temperature of the material piled on each stage during the experiment is shown in Table 6.

1. When water was added to the material so that its moisture content became higher than 60%, the material could not retain it and a black water with a foul odor drained from the outlet.

2. The end product was black, sludge-like, and smelly. Its temperature scarcely rose, and the decomposition seemed to be incomplete. Therefore, no chemical analysis was made.

3. It was impossible to dry the material piled up in the digester with the natural draft coming in through the air vent of the digester.

Experiment 7.—Experiment 7 was made with material, the moisture content of which had been raised to 60% by the addition of water. The conditions were: Continuous flow; 2 days detention and a feed rate of 0.05 cu m per hr at the beginning and 0.075 cu m per hr after 24th hour. The material was refuse 2 parts and end product 1 part. The moisture content was 60% (water was previously added to material). The temperature of the material piled on each stage of the digester during the experiment is indicated in Table 7. The results of the chemical analysis is shown in Table 8.

TABLE 7.—(Exp. 7)[a,b]

	Top stage	2nd stage	3rd stage	4th stage
Maximum	51	46	45	38
Mean	45.8	42	38.4	36.6

[a] All values in degrees centigrade.
[b] Mean ambient temperature: 30°C.

TABLE 8.—CHEMICAL ANALYSIS (Exp. 7)[a]

	Moisture, percentage	Ash, percentage	Volatile matter, percentage	Carbon, percentage	Nitrogen, percentage	C/N, percentage
Raw Material	60.3	60.9	39.1	21.7	0.83	26.2
End Product	52.1	64.4	35.6	19.8	0.76	26.0

[a] The raw material analyzed in this case was a mixture consisting of 2 parts ground refuse and 1 part end product. The end product was sludge-like and smelly.

Experiment 8 (Effect of intermittent operation).—In the previous experiments continuous flow operation had been adopted. In experiment 8, however, the operation was suspended for 8 hr at night, in order to ascertain the effects on the material in the digester. The conditions were: Duration of operation: 7.00 a.m. - 11.00 p.m. (16 hr per day); the detention period was 24 hr (rate of revolution of the central axis of the digester of 1/8 per min); the rate of feed was 0.098 - 0.131 cu yd per hr. Water was added so that the moisture content would be 60%. There was no reseeding.

The depth and the temperature of the material piled in the digester during the experiment are noted in Table 9 and the chemical analysis in Table 10.

Experiment 9 (Effect of seeding with patented Earp - Thomas No. 1. bacteria).—Experiment 9 was made with ground refuse that had been inoculated with Earp-Thomas No. 1 bacteria. The duration of the test was from

August 1, 1955 - August 6, 1955. The conditions were: Duration of operation: 7.00 a.m. - 11.00 a.m. (16 hr per day); 64 hr detention; a feed rate of 0.098 cu yd per hr. Water was added to the material so as to raise its moisture content to 60%.

At first, Earp-Thomas bacteria, lime, molasses and soil were mixed together at the ratio of 1, 8, 16 and 1,600, respectively, and cured for 4 days

TABLE 9.—(Exp. 8)[a]

		Top stage	2nd stage	3rd stage	4th stage
Depth of material piled, in centimeters	Maximum	25	23	25	30
	Mean	21.7	17.5	19.0	19.2
Temperature, in °C	Maximum	51	46	40	42
	Mean	45.0	40.8	36.8	36.1

[a] Mean ambient temperature: 29°C.

TABLE 10.—CHEMICAL ANALYSIS (Exp. 8)[a]

	Moisture, percentage	Ash, percentage	Volatile matter, percentage	Carbon, percentage	Nitrogen, percentage	C/N, percentage	pH
Raw Material	44.9	53.8	46.2	25.7	0.89	28.8	8.5
End Product	52.6	64.7	35.3	19.6	0.87	22.5	8.7

[a] The raw material sample was taken prior to the addition of water.

TABLE 11.—(Exp. 9)

		Top stage	2nd stage	3rd stage	4th stage
Depth of material, in centimeters	Maximum	25	20	20	20
	Mean	22.5	17.5	17.5	20
Temperature, in °C	Maximum	50	42	39	39
	Mean	46	41	37	37

in a wooden box. An ounce of this prepared cultive was mixed with 1 ton of garbage.

The depth and the temperature of the material piled on each stage of the digester during the experiment are shown in Table 11. The results of the chemical analysis are shown in Table 12.

Experiment 10 (Effect of air forced into the digester).—The duration of experiment 10 was August 15-August 22, 1955. The detention was for 64 hr but the actual running time of the digester was 48 hr, because between 11 p.m. and 7 a.m. there was no plowing, no feed and no forced air. The feeding rate

was 0.098 cu yd per hr. The depth of material and the maximum and mean temperatures are shown in Table 13 and the chemical analysis in Table 14.

1. No effects of the forced air were observed. This would mean that the pile of the ground material was porous and, therefore, contained sufficient air for composting.

2. At night when the plowing was suspended completely, the temperature went up. The maximum temperature given above was the temperature registered at 7 a.m., or immediately before the resumption of operation of the digester.

TABLE 12.—CHEMICAL ANALYSIS (Exp. 9)

	Moisture, percentage	Ash, percentage	Volatile matter, percentage	Carbon, percentage	Nitrogen, percentage	C/N, percentage	pH
Raw Material	56.3	54.5	45.5	25.2	0.93	27.1	6.7
End Product	44.6	64.4	35.6	19.8	0.83	23.9	7.3

TABLE 13.—(Exp. 10)

		Top stage	2nd stage	3rd stage	4th stage
Depth of Material, in centimeters	Maximum	20	18	18	25
	Mean	19	16	16	21
Temperature, in °C	Maximum	49	53	49	50
	Mean	44	44	43	43

TABLE 14.—CHEMICAL ANALYSIS (Exp. 10)

	Moisture, percentage	Ash, percentage	C/N, percentage
Raw material	39.8	47.6	29.1
End product	40.5	55.2	24.5

Experiments 11, 12, and 13 (Effect of addition of nightsoil to refuse).—The duration of experiment 11 was 64 hr but the actual running time of the digester was 48 hr, because between 11 p.m. and 7 a.m. there was no plowing or feed. The feeding rate was 0.098 cu yd per hr and the mixing ratios were as follows:

Refuse (5 parts) and nightsoil (2 parts), by wet weight. (Aug. 26 - Aug. 27)
Refuse (2 parts) and nightsoil (1 part), by wet weight. (Aug. 29 - Sept. 10)

Depth of the material and the maximum and mean temperatures are shown in Table 15. The end product had no nightsoil odor at all, was quite dry and

indistinguishable from compost made from refuse to which no nightsoil had been added.

Experiment 12 had a duration from September 12, 1955 - September 17, 1955. The detention period and the charging amount were the same as in experiment 11. The mixing ratio was 2 parts refuse and one part nightsoil, by wet weight. Depth of the material and the maximum and mean temperatures are shown in Table 16. The initial and final natures of the material are indicated in Table 17.

TABLE 15.—(Exp. 11)[a]

		Top stage	2nd stage	3rd stage	4th stage
Depth of material, in centimeters	Maximum	25	21	20	25
	Mean	18.2	13.3	15.8	19.0
Temperature, in °C	Maximum	50	44	43	45
	Mean	42.3	39.2	29.6	38.0

[a] Mean ambient temperature: 27.6°C.

TABLE 16.—(Exp. 12)[a]

		Top stage	2nd stage	3rd stage	4th stage
Depth of Material, in centimeters	Maximum	15	12	15	17
	Mean	13	11	15	15
Temperature, in °C	Maximum	58	49	47	42
	Mean	52.5	44.8	43.0	39.0

[a] Mean ambient temperature: 27.8°C.

TABLE 17.—INITIAL AND FINAL NATURES OF THE MATERIAL (Exp. 12)[a]

	Moisture, percentage	Ash, percentage	Organic matter, percentage	C/N, percentage
Initial	58.8	41.5	58.5	32.5
Final	51.8	56.0	44.0	24.6

[a] The end product had no nightsoil odor.

The mixing ratio for experiment No. 13 was 4 parts refuse and 3 parts nightsoil, by wet weight. This experiment began on September 19, 1955, and lasted till September 24, 1955. In the course of the experiment, however, the driving belt of the feeder elongated excessively and had to be replaced by a chain drive. Continuous operation was interrupted by this breakdown, however, the possibility of adding 3 parts of nightsoil to 4 parts of raw refuse with the moisture of 30% - 35% (by wet weight) was ascertained.

Experiment 14 (Effect of regrinding raw refuse).—Experiment 14 lasted from September 26 to October 1, 1955. The fineness of the material ground by the horizontal swing hammer grinder was; refuse, 12mm - 25mm; straw and paper, 150mm - 200mm or more. The result of regrinding this material brought no noticeable improvement on its fineness. It was therefore, impossible, with the existing grinder, to determine the effects of the fineness of raw material on the composting process.

Experiment 15 (A study of the activities of micro-organisms during composting).—Experiment 15 lasted from October 3, 1955, to October 10, 1955. The

TABLE 18.—(Exp. 15)[a]

		Top stage	2nd stage	3rd stage	4th stage
Depth of Material, in centimeters	Maximum	25	20	20	25
	Mean	20.1	16.1	17.9	21.6
Temperature, in °C	Maximum	55.0	50.0	45.0	37.0
	Mean	48.0	41.5	37.1	29.4

[a] Mean ambient temperature: 22.6°C.

TABLE 19.—CHEMICAL ANALYSIS (Exp. 15)

	pH	Moisture, percentage	Ash, percentage	Volatile matter, percentage	Nitrogen, percentage	Ammoneous Nitrogen, percentage
Raw material	6.5	52.0	42.9	57.1	0.98	0.006
Top stage	6.8	44.0	48.8	51.2	0.97	0.040
2nd stage	7.0	44.7	48.0	52.0	0.99	0.036
3rd stage	7.1	40.6	53.2	46.8	0.85	0.022
4th stage	7.0	41.5	54.8	45.2	1.00	0.025
End Product	7.0	39.3	54.6	45.4	1.02	0.010
End Products piled in windrows 5-7 days	7.5	32.0	58.2	41.8	0.96	0.040

detention was for 64 hr but the actual running time of the digester was 48 hr, because between 11 p.m. and 7 a.m. there was no plowing and no feed. Ground refuse feed was at the rate of 0.131 cu yd per hr.

Depth of the material and the maximum and mean temperatures are shown in Table 18 and the chemical analyses in Table 19. Percentages of volatile matter, and moisture and pH at various stages in Fig. 8.

Decomposition of Crude Fiber During Composting.—If the ash content is indicated by 100, the organic matter content and the crude fiber content vary during composting as shown in Table 20. The ash content does not change in the course of composting.

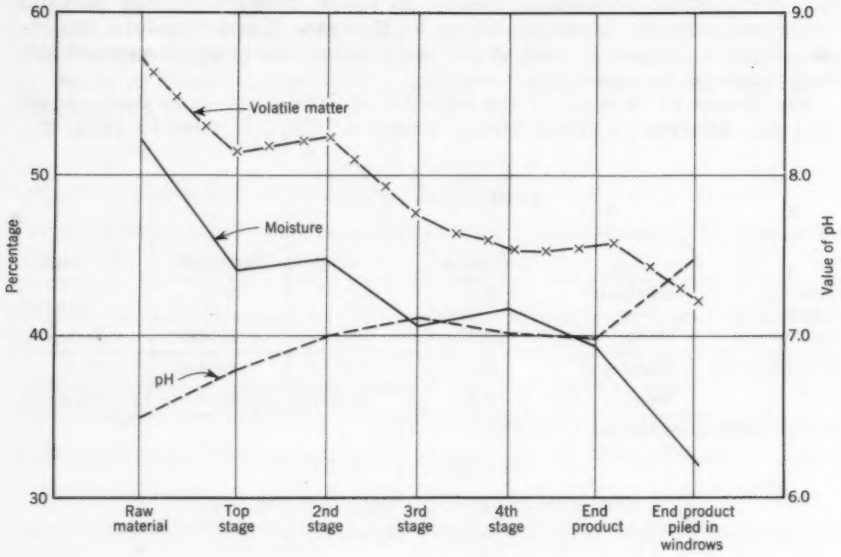

60
50
40
30
9.0
8.0
7.0
6.0
Percentage
Value of pH
Volatile matter
Moisture
pH
Raw material
Top stage
2nd stage
3rd stage
4th stage
End product
End product piled in windrows

FIG. 8

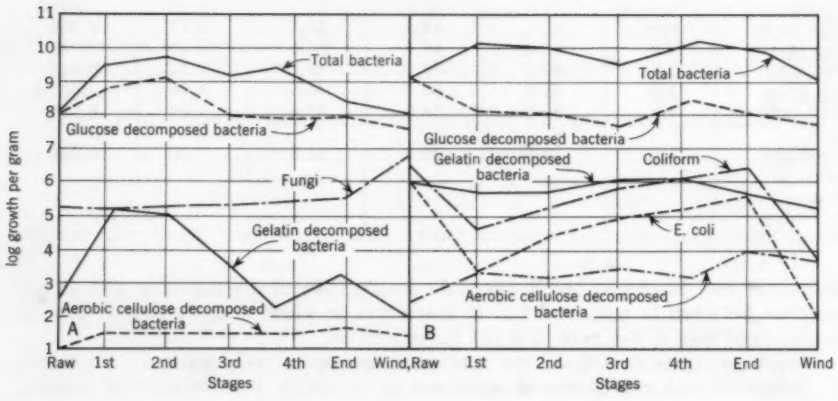

log growth per gram
Total bacteria
Glucose decomposed bacteria
Fungi
Gelatin decomposed bacteria
Aerobic cellulose decomposed bacteria
A
Raw 1st 2nd 3rd 4th End Wind
Stages
Total bacteria
Glucose decomposed bacteria
Gelatin decomposed bacteria
Coliform
E. coli
Aerobic cellulose decomposed bacteria
B
Raw 1st 2nd 3rd 4th End Wind
Stages

FIG. 9

Table 21 shows the percentage decomposition (in A and B courses) of the preceding components, computed on the basis of Table 20. The remarkable fact shown by Table 21 is that when the end product from the digester was piled in windrows, the decomposition of crude fiber was 23.3% while that of other organic matters was only 3.99%.

The distribution of different physiological groups of micro-organisms at each stage of composting is shown in Table 22.

Table 22 is presented graphically in Fig. 9. It is presumed that thermophilic glucose decomposing bacteria played the principal role on the top and 2nd stages. It also is presumed that when the end product was piled in windrows, fungi grew and decomposed cellulose.

Experiment 16 (Effect of addition of nightsoil to refuse).—The mixing ratio for experiment 16 was 2 parts refuse and 1 part nightsoil. The test lasted

TABLE 20

		Ash	Total organic matter	Crude fiber	Organic matters other than crude fibers
	Raw material	100	133	50.9	82.1
A	End product from digester	100	83	40.4	42.6
B	Above end product piled in windrows 5-7 days	100	71.9	31.0	40.9

TABLE 21

		Total organic matter, percentage	Crude fiber, percentage	Organic matter other than crude fiber, percentage
A	In digester	37.6	20.3	48.0
B	In windrows	13.6	23.3	3.99

from October 11, 1955 - October 15, 1955. The detention was the same as Experiment 11. The depth of the material and the maximum and mean temperatures are shown in Table 23 and the initial and final nature of the material in Table 24.

Experiment 17 (Effect of partial recirculation of the end product).—The recirculation ratio for experiment 17 was one part raw refuse and 0.2 parts end product, by weight. The test lasted from October 17, 1955 - October 22, 1955. Detention time was 2 days (continuous flow).

Depth of the material and the maximum and mean temperatures are shown in Table 25. As compared with the preceding experiments, the temperatures on each stage of the digester were higher in experiment 17.

Experiment 21. (Effects of the quantity of vegetables contained in raw material).—The mixing ratio for experiment 21 was one part vegetables to 2 parts refuse by volume. The test lasted from November 28, 1955 - December

TABLE 22.—MICRO-ORGANISM DISTRIBUTION[a]

		Raw Material	Top stage	2nd stage	3rd stage	4th stage	End product	End product piled in windrows
A	Total bacteria	$1.8x10^8$	$5.4x10^9$	$9.2x10^9$	$1.7x10^9$	$3.5x10^9$	$3.5x10^8$	$1.7x10^8$
	Glucose decomposing bacteria	$1.8x10^8$	$9.2x10^8$	$1.8x10^9$	$1.7x10^9$	$1.4x10^8$	$1.8x10^8$	$4.9x10^7$
	Gelatin decomposing bacteria	$3.5x10^2$	$1.6x10^5$	$1.6x10^5$	$9.2x10^3$	$2.4x10^2$	$2.4x10^3$	$1.3x10^2$
	Aerobic cellulose decomposing bacteria	$1.1x10^1$	$3.3x10^1$	$3.3x10^1$	$3.3x10^1$	$3.3x10^1$	$4.9x10^1$	$3.3x10^1$
B	Total bacteria	$1.7x10^9$	$1.6x10^{10}$	$9.2x10^9$	$3.5x10^9$	$1.6x10^{10}$	$5.4x10^9$	$9.5x10^8$
	Glucose decomposing bacteria	$1.7x10^9$	$1.7x10^8$	$1.4x10^8$	$4.9x10^7$	$3.5x10^8$	$7.9x10^7$	$4.9x10^7$
	Gelatin decomposing bacteria	$9.2x10^5$	$5.4x10^5$	$5.4x10^5$	$1.6x10^6$	$1.6x10^6$	$3.5x10^5$	$1.3x10^5$
	Aerobic cellulose decomposing bacteria	$3.3x10^2$	$3.4x10^3$	$2.4x10^3$	$3.5x10^3$	$1.3x10^3$	$9.2x10^3$	$3.3x10^3$
	Fungi	$3.2x10^5$	$2.0x10^5$	$2.8x10^5$	$2.5x10^5$	$3.95x10^5$	$4.1x10^5$	$9.1x10^6$
	Coliform	$4.9x10^6$	$4.5x10^4$	$2.2x10^5$	$4.9x10^5$	$1.3x10^6$	$3.3x10^6$	$3.3x10^3$
	E. Coli.	$7.9x10^5$	$2.8x10^3$	$2.2x10^4$	$7.0x10^4$	$1.7x10^5$	$2.7x10^5$	$9.2x10^1$

[a] (Organisms; per gram, wet weight basis).

Notes: (A at 55°C
(B at 37°C
The fungi were obtained by Waksman's culture. (plate culture) at 27-30°C.
The numerical figures for those other than fungi were obtained by the "most probable number method."
The above data were compiled by the Osaka Hygienic Laboratory at the request of the writer.

TABLE 23.—(Exp. 16)[a]

		Top stage	2nd stage	3rd stage	4th stage
Depth of material, in centimeters	Maximum	20	23	20	25
	Mean	18.7	18.4	18.6	21.6
Temperature, in °C	Maximum	59	57	51	43
	Mean	51.3	44.6	39.4	31.6

[a] Mean ambient temperature: 18.8°C.

3, 1955. The detention period and the feed rate were the same as those of experiment 15.

The moisture of the raw material was 75% and the nitrogen content was 1.63%.

Due to the excess of moisture, the end product had a putrid ordor.

Composting in Windrows.—At the same time the previously described high-rate composting experiments were being made, a separate investigation of windrows composting was carried out.

TABLE 24.—INITIAL AND FINAL NATURES OF THE MATERIAL[a]

	Moisture, percentage	Organic matter, percentage	Ash, percentage	C, percentage	N, percentage	C/N, percentage	pH
Initial	60.7	47.7	52.3	26.5	0.92	28.8	6.6
Final	43.7	43.9	56.1	24.4	1.12	21.8	7.1

[a] The end product had no nightsoil odor.

TABLE 25.—(Exp. 17)[a]

		Top stage	2nd stage	3rd stage	4th stage
Depth of material, in centimeters	Maximum	20	22	25	22
	Mean	15.8	19.0	20.6	22.6
Temperature, in °C	Maximum	65	64	50	40
	Mean	55.7	52.2	42.9	35.7

[a] Mean ambient temperature: 19°C.

TABLE 26.—COMPOSITION OF RAW MATERIAL[a]

Vegetable and putrescible	Paper	Soil	Straw	Metal	Porcelain	Wood	Textile	Rubber
50.6	6.4	25.2	2.1	1.0	5.2	6.3	2.5	0.7

[a] All in percentages.

Experiment 1.—The physical compostion of raw material for experiment 1 is shown in Table 26. The test lasted from May 19, 1955 - June 7, 1955. The volume of the material piled was 1.31 cu yd (31 in. high). The material was turned on the 2nd, 6th, 7th, 8th, 9th, 12th, 13th, 14th, 17th, and 18th days. The results of the chemical analysis are shown in Table 27 and the variation in temperature in Table 28.

Experiment No. 2.—Experiment 2 lasted from July 16 to August 4, 1955. The volume of the raw refuse was 1.31 cu yd (31 in. high). Water was added to the material to raise the moisture content to 60%. There was no turning of the

material. Temperature variation during composting is shown in Table 30 and chemical analysis in Table 29.

Aeration.—Usually temperature is regarded as the measurement required to know to what degree material under treatment is composted. It is very doubtful, however, that temperature is a good measurement for this purpose, because a high temperature of the material in the process of composting does

TABLE 27.—CHEMICAL ANALYSIS (Exp. 1)[a]

	Moisture	Ash	Volatile matter	Carbon	Nitrogen	C/N
Initial	49.7	49.6	50.4	28.6	0.95	29.4
5th day	40.8	52.1	47.9	26.6	1.02	26.2
7th day	39.2	55.5	44.5	24.9	0.98	25.6
9th day	32.3	58.6	41.4	23.1	1.01	22.6
12th day	23.2	58.3	41.7	23.1	0.94	24.5
14th day	24.0	62.2	37.8	21.0	0.93	22.7
17th day	19.5	63.8	36.2	20.0	0.90	22.4

[a] All in percentages.

TABLE 28.—TEMPERATURE VARIATIONS (Exp. 1)

Date		May 19 20 21 22 23 24 25 26 27 28 29 30 31	June 1 2 3 4 5 6 7
Depth of pile, in centimeters	5	29 61 56 / 54 55 55 58 37 28 / 44 32	44/ /55 35 / 32
	10	30 63 62 / 61 57 60 60 44 38 / 54 41	54/ /60 45 / 40
	15	30 62 66 / 66 60 66 65 52 45 / 58 48	59/ /65 44 / 43
	20	30 60 60 / 66 65 67 65 59 53 / 58 49	64/ /65 45 / 43

TABLE 29.—CHEMICAL ANALYSIS (Exp. 2)

Time in days	Moisture, percentage	Ash, percentage	Volatile matter, percentage	Carbon, percentage	Nitrogen, percentage	C/N, percentage	pH
Initial	60.5	56.7	43.3	24.1	0.82	29.3	6.7
5th	50.2	59.1	40.9	22.7			7.3
10th	45.7	62.9	37.1	20.6			7.5
15th	49.0	62.7	37.3	20.1			7.6
20th	41.5	63.2	36.8	20.4	0.92	22.2	7.2

not necessarily mean that the decomposition of material is proceeding well. If the material is piled up and left alone, its temperature goes up. In addition, the material has the capacity to retain heat. Therefore, once its temperature rises, that high temperature will be maintained, even though the decomposition of the material is actually low. For such decomposition it is necessary to provide an ample and continuous supply of oxygen. An experiment made at

Michigan State University shows that the oxygen contained in garbage in windrows decreased to 3% at a point 8 in. below the surface in 10 hr after this pile of garbage was turned over.

The following describes a series of experiments that were made in order to find out the proper amount of aeration for the composting of refuse. The study on this subject has not been concluded (as of 1961).

The bins used for experiments were 55-gal drums, each of which had an air inlet and outlet. Material used was 165 lb of ground raw refuse. Aeration was 0.3 cu ft per min for Bin A and 0.6 cu ft per min for Bin B. Results are as indicated in Table 31 and Fig. 10.

TABLE 30.—VARIATION OF THE TEMPERATURE (Exp. 2)

Date		July 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31	Aug. 1 2 3 4
Number of days		1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16	17 18 19 20
Depth below surface of pile, in centimeters	5	38 / 48 52 50 50 42 49 / 51 49 45 44 45 42 /	40 37 36 35
	20	42 / 65 64 61 63 58 62 / 58 56 57 55 52 51 /	46 35 44 42

TABLE 31.—AERATION RESULTS

	Period	Quantity of air, in cubic feet per minute	Quantity of air per 100 grams organic matter per hour, in cubic centimeters	Stage	pH	Moisture, percentage	Ash, percentage	Volatile matter, percentage	Total quantity, kilograms	Quantity of decomposed organic matter, kilograms
A	6/4-6/15	0.3	3900	initial	6.2	70.7	41.8	58.2	75.0	
				final	6.9	70.9	45.7	54.3	67.2	2.2
B	7/3-7/16	0.6	7250	initial	5.2	72.7	32.7	67.3	75.0	
				final	7.6	71.7	46.5	53.5	54.5	5.6

It is evident that a high temperature was maintained longer in A than in B (Fig. 10). However, decomposition was more active in B than in A. This fact indicates the importance of aeration in the composting of refuse.

ANALYSIS OF EXPERIMENTS

Degree of Decomposition.—The degree of decomposition in the process of continuous flow digestion is expressed as

$$\left(1 - \frac{\frac{b}{a^{r}}}{\frac{b}{a^{r}}}\right) \times 100$$

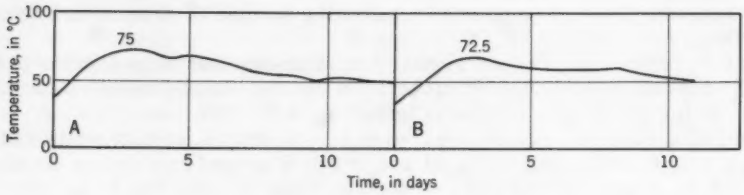
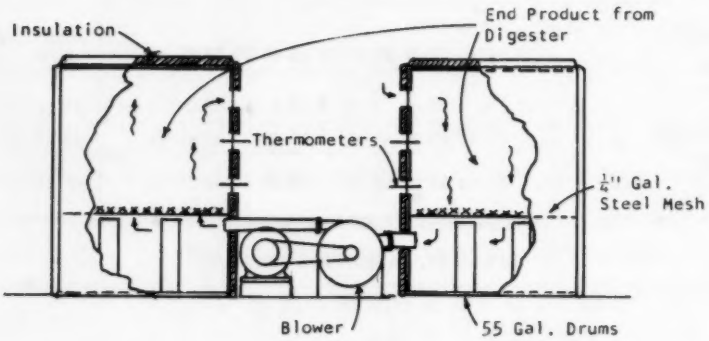
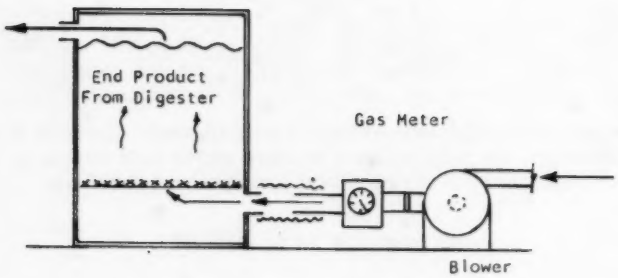


FIG. 10



(a) DUAL BIN ARRANGEMENT



(b) SINGLE BIN ARRANGEMENT

FIG. 11.—SCHEMATIC DIAGRAM OF CURING BIN APPARATUS

in which a is the percentage of ash of the end product, a' denotes the percentage of ash of the raw material, b is the percentage of volatile matter of the end product, and b' refers to the percentage of volatile matter of the raw material. The values are given in the summary tables previously noted for the experiments.

Moisture Content.—The moisture content of the material was especially high in experiments 6 and 21. In these experiments a black and smelly drainage liquid was observed and the end product was mud-like, smelly and proved to be anaerobic.

It is presumed that in experiments 11, 13, and 20 also, the moisture content of the material was high because the inside temperature of the digester was lower in these experiments than in the others. For the pilot plant it was assumed that the critical moisture content of the material was 60%. Of course, if a digester which had thicker walls, and thus prevented the heat from escaping, were used, material of a higher moisture content could be treated.

Aeration.—It is known that when material is composted in windrows, the amount of natural aeration is small and oxygen runs short rapidly, causing decomposition to be slow. In each of the windrow composting experiments, the material maintained a temperature of 60°C for approximately 10 days. When the results of these experiments are compared with the fact that the decomposition of the material in the digester was nearly completed in 20 hr, the necessity for aeration is obvious. Because too much air is apt to cool the material, it is important to pay attention to this problem in the composting operation.

When the amount of aeration is identical with the required quantity of oxygen, the decompostion of the material by mesophilic bacteria takes place first and then high temperature caused by this decomposition provides a favorable environment for thermophilic bacteria that begin their activities in the later stage. However, some portions of the material will attain a temperature that is high enough to destory bacteria and it is supposed that in such conditions thermophilic bacteria take some time for recovery. The most favorable conditions for composting will be produced when (1) the amount of aeration is large enough to deprive the material of the degree of heat capable of destroying bacteria, and (2) the amount of aeration is capable of keeping the material at the temperature that is favorable for the growth of thermophilic bacteria.

Digester.—The first problem to be studied with regard to the efficiency of the pilot plant was its ability to keep heat because it was presupposed that with this plant considerable heat would escape. The temperature inside the digester seldom rose over 60°C, the stabilization of material in the digester was incomplete, and the temperature of the end products, even after discharged from the digester, kept rising. In the digester, the coliform groups in the material decreased initially, however, they showed a tendency to increase on the lower decks.

The decomposition of material in the digester consisted mostly of the decomposition of carbohydrates and proteins. Thermophilic bacteria played the principal role in this decomposition, which took place mostly on the 1st deck and the 2nd deck.

In the digester, the decomposition of organic matter was approximately 30% and the material turned into a good quality humus. Nevertheless, the material did not attain the temperature necessary for destroying all pathogens,

specifically, 60°C. The temperature was especially low on the 3rd deck and the 4th deck because vapor goes up from the lower decks towards the upper decks. At the same time, the depth of material piled on each deck was too small, and the drying seemed to be taking place more or less on the lower decks. These are regarded as unique characteristics of this digester.

It seems that the most influential factors in high-rate composting are the plowing and the mixing of the material. Further studies must be made on the mechanical mixing features of the composting plant.

When the end products discharged from the digester are piled up, the temperature can be maintained at 60°C, which is sufficient to destory all pathogens. Therefore, a necessary feature of high-rate composting is the investigation of the curing process (which is explained later) and also the combining of the mechanical digester and the curing process.

The experiments made it clear that it was impossible to prevent bad odors in the windrow composting of refuse. On the other hand, the composting operation with the digester proved to be entirely odorless. In addition, the mechanical digester requires little space. Therefore, because no large space is available for composting in any of the cities of Japan, the authorities of any of these cities will welcome the mechanical composting process.

CURING EXPERIMENTS

Preliminary Experiments.—In accordance with the advice of J. R. Snell, who pointed out the necessity of curing the end products from the digester for secondary digestion, curing experiments, were conducted, with two 55-gal bins as illustrated in Fig. 11(a).

Details of Experiments.—Experiments were also conducted with a single bin as illustrated in Fig. 11(b).

Results.—The results of the experiments are shown graphically in Fig. 12. (The solid line represents the temperature curve of the upper part of the material in the bin, and the dotted line shows that of the lower part of the bin.) Experimental conditions are shown in Table 32.

Experiment 1.—As fresh air entered Bin B, the temperature of the material in it did not exceed 20°C, while the temperature of the material in Bin A into which the air from Bin B was forced, did not exceed 50°C. Both of these results show the cooling effect of air and are considered to have been caused by too much aeration. B→A and A→B show the direction of aeration.

Experiment 2.—The temperature of the material in Bin B into which fresh air was forced did not exceed 13°C, while that of Bin A was below 30°C.

Experiment 3.—Bin A contained the end product from the digester only. Bin B contained a mixture of such end products and raw garbage. The lids of both bins were left open. Consequently, the upper part of the material was cooled, and its temperature rose more slowly and dropped more rapidly as compared with the lower part of the material in both bins.

Experiment 4.—The amount of aeration was less than in experiments 1 and 2 and the temperature exceeded 50°C.

Experiment 5.—In experiment 5 the temperature did not exceed 20°C, probably due to too much aeration.

Experiment 7.—During the first period when the bins were not aerated, the temperature scarcely rose. However, after the commencement of aeration,

the temperature went up slowly. This gradual rise of temperature was considered to have been caused by the cooling effect.

Remarks.—It appears that when the temperature of the outdoor air is low, the cooling effect due to air is large and the amount of aeration should be limited still further.

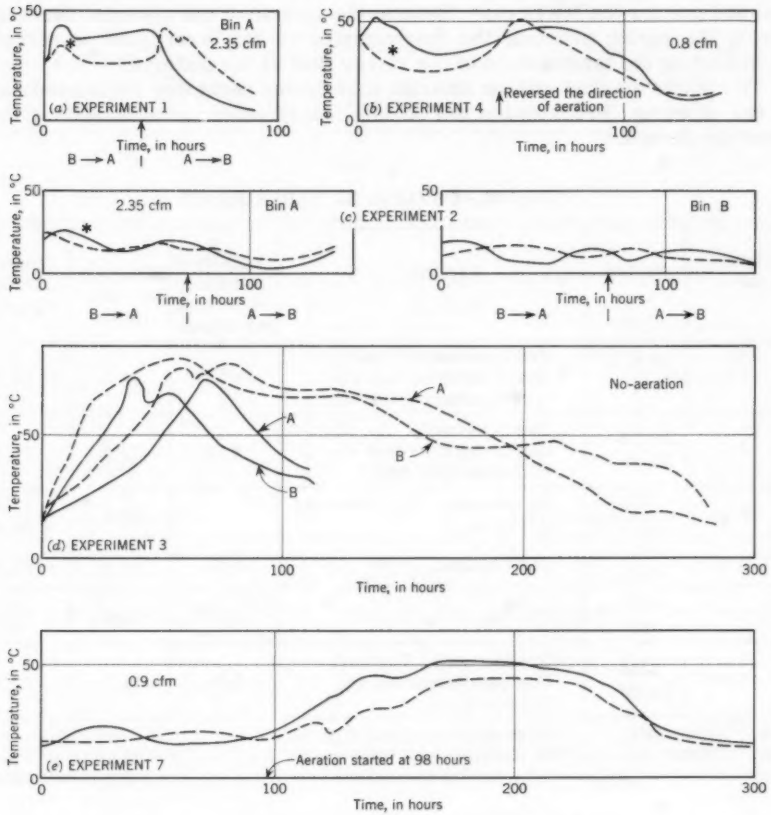


FIG. 12

The points marked with an asterisk (*) in the graphs show the temperatures registered early in the morning when temperatures are affected by the low temperature of the outdoor air during the night.

The temperature of the material in the bin was higher in experiments 4 and 7 when a smaller quantity of air was supplied.

Final Experiments (Effect of rate of aeration and curing).—

Introduction.—Because the preliminary experiments proved that the rate of aeration was a limiting factor in the curing process, curing effects were examined at various rates of air supply. End products from the digester were used as material.

Details of Experiments (See Table 33, Fig. 13 and Fig. 14).—Fig. 13 shows the variation of CO_2 in exhaust gas. In the case of Experiment 1 CO_2 is always 12% to 14%. Two thermometers were used, one for the upper part of the material and one for the lower part. Because the volume of the material decreased during the curing process, the thermometer for the lower part is regarded as indicating the temperature of the larger part of the material.

The points marked with an asterisk show the temperatures registered early in the morning. Probably the low outdoor temperature again caused the temperature change.

TABLE 32.—OUTLINE OF EXPERIMENTS

Experiment	Period	Conditions	Quantity of aeration, cubic feet per minute	Material fed, kilogram	Product taken out, kilogram
1	12/2 - 12/17	Two bins used. The direction of aeration was reversed after 2 days.	2.35	97.3	87.1
2	12/19- 12/25	Two bins used. The direction of aeration was reversed after 3 days.	2.35	73.9	71.1
3	1/28- 2/9	No aeration. The lids were left open.		85.0	A 73.5[a] B 69.4
4	2/4 - 2/21	One bin used.	0.8	72.8	65.0
5	2/25- 3/2	One bin used.	1.5	75.6	72.7
7	3/10- 3/22	One bin used. The aeration was suspended for 98 hr at first.	0.9	71.7	62.7

[a] A. . .(Material). . . . .End product from the digester.
B. . .(Material). . . . .End product from the digester 3 parts: ground garbage 1 part.

Analysis.—Figs. 13 and 14 clearly show that the quantity of aeration is a very influential factor. As had been supposed, the temperature rose little and little decomposition took place in the experiments in which there was almost no aeration. The amount of aeration that was most favorable for both temperature rise and decomposition was 0.26 cu ft per min, the second best being 0.41 cu ft per min. In the former case in which a high temperature was maintained the longest, decomposition was the highest.

It appears that the greater the aeration the sharper the temperature drop, thus clearly showing the cooling effect of air. Although the water content of the material decreased to some extent, the percentage moisture, in such an enclosed curing bin as this, did not decrease satisfactorily even when much

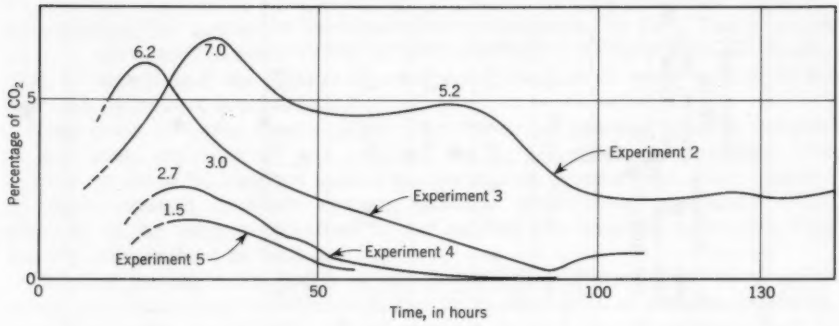

Percentage of CO_2
5
0
0
50
100
130
Time, in hours
6.2
7.0
5.2
3.0
2.7
1.5
Experiment 2
Experiment 3
Experiment 4
Experiment 5

FIG. 13

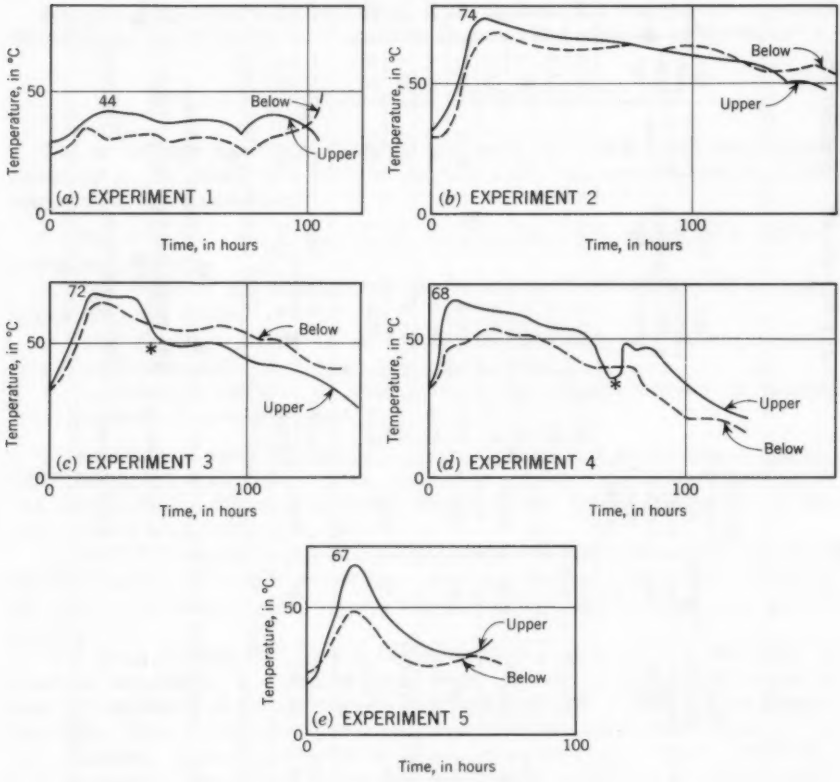

Temperature, in °C
50
0
0
100
Time, in hours
44
Below
Upper
(a) EXPERIMENT 1
74
Below
Upper
(b) EXPERIMENT 2
72
Below
Upper
(c) EXPERIMENT 3
68
Upper
Below
(d) EXPERIMENT 4
67
Upper
Below
(e) EXPERIMENT 5

FIG. 14

TABLE 33.—FINAL EXPERIMENTS

Exp. No.	Period	Range of Ambient temperature °C	Quantity of air, cfm	Quantity of air per 100gm organic matter, in cc per hr		Total quantity, in kilograms	Moisture, percentage	Volatile matter, percentage	Ash, percentage	Carbon, percentage	Nitrogen, percentage	C/N, percentage	Decrease of moisture, kilograms	Decomposed organic matter, in kilograms		pH
														A	B	
1	5/27-6/1	22 (18-25)	No-aeration		Initial	105.3	52.0	43.7	56.3	24.3	1.33	18.3				7.3
					Final	103.5	52.3	44.2	55.8	24.6	1.24	19.8	0.7[a]	1.1[b]	0.2	7.0
2	4/26-5/4	17 (13-20)	0.26	1910	Initial	101.7	48.1	50.8	49.2	28.2	1.36	20.8				7.1
					Final	83.5	48.7	44.9	55.1	25.0	1.45	17.3	9.8	8.4	7.6	7.2
3	5/21-5/26	19 (16-24)	0.41	3590	Initial	92.0	53.2	45.3	54.7	25.2	1.27	19.8				7.8
					Final	81.0	52.6	40.5	59.5	22.5	1.30	17.3	6.4	4.6	4.0	7.9
4	5/10-5/15	5/10-5/13,20 5/13-5/15,15	0.73	5680	Initial	83.9	47.4	49.9	50.1	27.7	1.38	20.1				6.9
					Final	75.6	45.5	46.5	53.5	25.8	1.42	18.3	5.4	2.9	2.8	7.0
5	5/18-5/21	19 (16-24)	1.47	13500	Initial	82.9	52.0	46.3	53.7	25.7	1.34	19.2				6.4
					Final	76.9	52.6	43.8	56.2	24.4	1.30	18.8	2.8	3.2	2.4	8.1

[a] Decrease of moisture = $\frac{\text{Initial amounts} \times \text{initial moisture \%}}{100} - \frac{\text{Final amounts} \times \text{Final moisture \%}}{100}$

[b] Decomposed organic matter A = Initial amounts - final amounts - decreased moisture

$$B = \left\{ \text{Initial amounts} \times \frac{(100 - \text{initial moisture \%})}{100} \times - \frac{\text{Initial volatile matter \%}}{100} \right\}$$

$$- \left\{ \text{Final amounts} \times \frac{(100 - \text{final moisture \%})}{100} \times - \frac{\text{Final volatile matter \%}}{100} \right\}$$

air was supplied, owing to the decrease of organic materials. However, it would be possible to maintain a high temperature and to destory pathogenic bacteria by adjusting the amount of aeration. C/N ratio dropped by 2 - 3. The nitrogen content decreased in many cases as evidenced by the ratio of nitrogen to ash. Fig. 13 shows that the ratio of decomposition reached its peak in 20 to 30 hr and then decreased gradually.

The most effective limiting factor for the curing process was the quantity of air; when too much air was supplied, the cooling effect was prounced. The amount of air to be supplied should be determined by considering the quantity of undecomposed, unstable organic matters, the outdoor temperature, the quantity of the oxygen contained in the exhaust gas (must be more than 5%), and the temperature in the bin.

Whether curing is necessary or not should be determined from the quantity of the unstable organic matter contained in the digester and with consideration also being given to the temperature at which the end product has been kept in the digester. It is not considered necessary to cure those products that have been kept at a high temperature in the digester long enough to destroy the pathogenic bacteria and decompose satisfactorily.

As Snell suggested, the installation of curing bins will shorten the required detention period in the digester and thus increase the capacity of the digester.

PROTO-TYPE HIGH-RATE COMPOSTING PLANT

The proto-type plant was designed and built according to the suggestions given by J. R. Snell, for WHO in August, 1955. The essential points of his suggestions were as follows:

1. The grinder, which was to be supplied by WHO, should be a vertical swing hammer type.
2. The outlet of the storage bin should be over 5 ft in diameter, in order to prevent arch action.
3. The digester should be a 3-stage horizontal type with continuously mixed and horizontally circulated flow on each deck.
4. A secondary digester (in which material would not be stirred but aerated from the bottom) should be used.

This proto-type plant through the primary digester was completed in August, 1956. In March, 1957, the curing bin and the piping for adding raw nightsoil and anaerobically digested nightsoil sludge to the ground garbage in the digester were completed. (Fig. 15).

General Description.—The composting plant was built within the compounds of the Takamatsu Incineration Plant, with its design based on the results of the pilot plant experiments and the support of WHO and the Japanese central government.

The primary digester is the mainstay of the plant and will stabilize, in common operation, a mixture of 20 tons of sorted, ground refuse and 10 tons of nightsoil or anaerobically digested nightsoil sludge during 2 days' detention. This digester is patterned after the Crane digester (T. A. Crane Ltd., Toronto, Canada) although it is not identical in all respects (Fig. 16). The secondary digester (a curing and storage bin) will finish the end product during 2 days' detention.

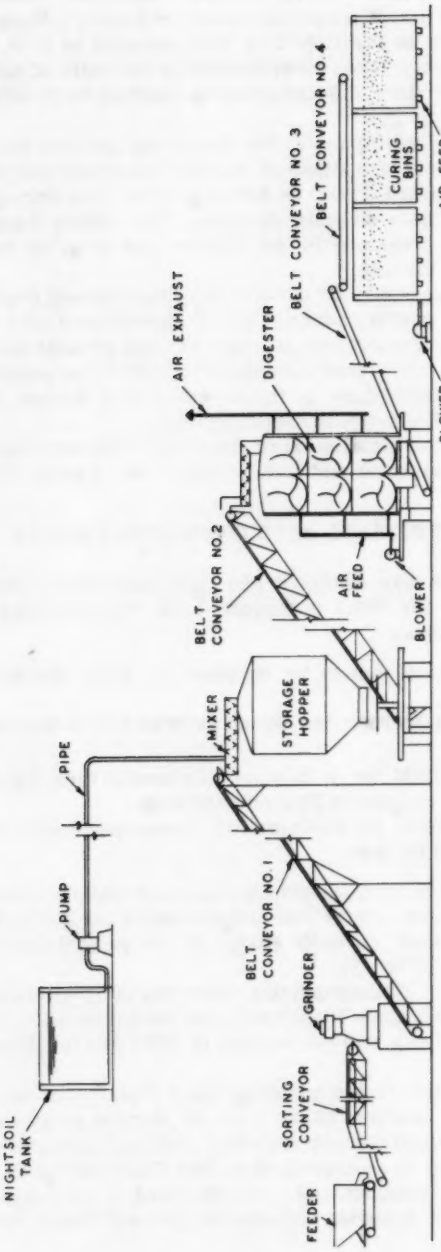


FIG. 15.—PROTO-TYPE HIGH-RATE COMPOSTING PLANT

Other items of equipment are as follows: feeder; conveyor for segregation; grinder; storage bin; mixer; and, conveying equipment between units.

Feeder.—The feeder is an apron conveyer which uniformly carries refuse from the bottom of the receiving bin to the succeeding conveyor, the rate being 26 cu yd per hr.

Conveyor for Segregation.—From the raw material on the conveyor, laborers remove glass, cans, wood and other non-compostable substances by hand-sorting. The width of the belt is 36 in. and the length 33.5 ft. The speed of this conveyor is 33 fpm.

Grinder.—The grinder, which was supplied by WHO, is the vertical swing hammer type, 1,800 rpm and driven by a motor of 100 hp. It has a grinding capacity of 26 cu yd per hr (7 tons per hr). The maximum diameter of refuse pieces fed is 5 in. The particle size of ground refuse is $\frac{1}{2}$ in. or less and the size of ground cardboard and straw is 2 in. or less.

Storage Bin and Table Feeder.—The feeder, conveyor for segregation, and grinder were designed to be operated continuously for 6 hr during daytime. Therefore, in order that ground material could be discharged into the primary mechanical digester continuously for 24 hr, storage facilities had to be provided for ground material. The storage bin has a capacity of 53 cu yd. The outlet on the table feeder is so fixed that the quantity of material coming-down may be decreased to 5 cu yd maximum per hr.

Primary Mechanical Digester.—The digester is an outdoor, 3-stage or deck, horizontal type with a continuously mixed flow. Each is divided by a partition. Each partitioned space has a center shaft to which a number of agitator blades are fitted at an angle of 120° so that raw material will be slowly stirred and moved. At the same time, 10% of the raw material on each deck is recirculated as seeding material. In this way the growth of mesophilic bacteria and thermophilic bacteria is accelerated. During the 2 days detention, organic wastes are stabilized and the oxygen necessary for this stabilization is supplied by an air blower. The digester is insulated with rock wool. Its dimensions and rating are as follows:

Width	10.5 ft
Length	25 ft
Height	13 ft
Effective capacity	115 cu yd
Rate of revolutions of the center shaft	1/5 per min
Motor	10 hp

The auxiliary apparatus consists of hand holes, sampling holes, air vents, air orifices, thermometers, operating decks, and so on.

Blower.—The specifications for the blower are as follows:

Amount of air	47 cu ft per min
Airpressure	8 in. water column
Motor	7.5 hp

Belt Conveyors.—The specifications for the belt conveyor are shown in Table 34.

Curing Bin.—The end product from the primary digester is carried to the curing bin by a conveyor. This bin is 35 ft long, 16 ft wide and 8 ft high and is divided into three sections. The end product is detained in this bin for approximately 48 hr and undergoes secondary digestion with the aid of the air forced

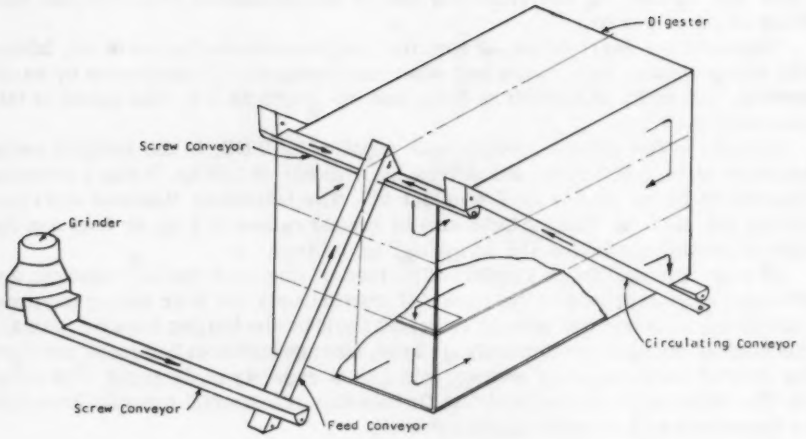


FIG. 16.—REVISED DIGESTER LAYOUT

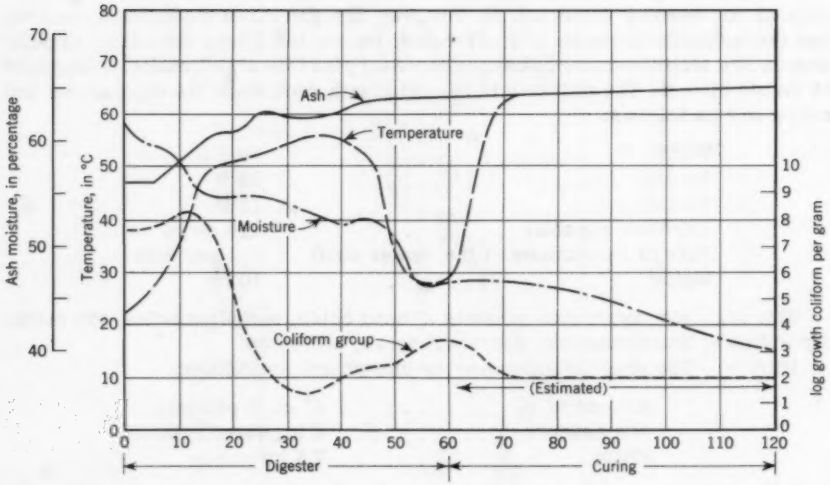


FIG. 17

in by a blower (800 cu ft per min). This bin functions not only as a secondary digester but also as a storage bin. The material cured in this way is sent out by truck in bulk.

In the operation of the proto-type plant, several obstacles were encountered.

1. The Grinding Equipment.—It was expected that the vertical swing hammer grinder could grind 7 tons of refuse an hour; however, its actual capacity has proved to be only 2-3 tons per hr, for the following reasons:

(a). The quantity of material that the sorting conveyor carries to the grinder varies widely and thus the total amount of the material fed to the grinder during the operating hours is less than the design quantity because there is no mechanism that gives continuous and uniform feeding to the sorting conveyor.

(b). The swing hammer blades wear rapidly and when worn out, the discharge outlet clogs. Vertical swing hammers of a modified type are being manufactured (1961).

TABLE 34.—SPECIFICATIONS FOR BELT CONVEYORS

	Width, in inches	Length, in feet	Speed, in feet per minute	Capacity, in cubic yards per hour
Between grinder and Mixer	16	47	130	18
Between table feeder and digester	12	39	130	9
Between digester and curing bin	12	28	130	9
Conveyor for distribution	12	29	130	9

2. The Storage Bin and the Table Feeder.—Because the outlet of the storage bin is shaped like an inverted cone, the stored material arches at the outlet when the bin is full and does not fall to the table feeder, which is installed beneath the outlet. With the present mechanism, it was almost impossible to secure the continuous discharge of material.

To alter the storage bin, many experiments must be conducted. At present, therefore, ground material is charged directly to the digester. In the revised plan, the storage bin will not be used.

3. The Digester.—The center shaft in each chamber of the digester has many plows and revolves at a speed of 1/5 rpm, stirring and pushing the material forward for composting.

The speed of the mixing plows was found to be too slow, and material accumulated thus creating a large force upon the plows and often breaking them. If plows of this type are required to play two roles, to mix the material and to push it forward at a slow speed, the device must be made stronger. The horizontal circulation of the material from one chamber to the other on each deck was not satisfactory, and the plows often broke at the point where the material turned sideways and passed from one chamber to the other.

In short, all the mechanical elements of the digester (the mixing plows, the center shaft, reduction gear, and so on, were too weak to mix and to push forward 20 tons of ground material a day. It became clear that the digester was, in reality, able to digest only 7 tons of ground material per day.

Results.—Fig. 17 shows the relations between the increase of ash and the reduction of the coliform groups in the material in the digester having the weak mechanical elements and in the curing bin, and the periods of digestion and curing.

TABLE 35.—MATERIAL ANALYSIS FROM PROTO-TYPE PLANT[a,b]

	(Dry basis)			
	Raw Material	Average	Final compost	Average
Moisture (Wet basis)	50-60	55	30-40	32.8 [c]
Ash	40-55	53.3	60-70	66.0
Volatile solids	45-60	46.7	30-40	34.0
Sand or clay	——		45-55	
Specific gravity	0.34-0.45		0.55-0.65	
Nitrogen (as N)	——		0.84-1.18	0.93
Phosphorus (as P_2O_5)	——		0.52-0177	0.62
Potash (as K_2O)	——		0.56-0.88	0.71
C/N ratio	25-34		17-23	

[a] Values have been determined on a dry basis, except moisture.
[b] All data in percentages.
[c] The moisture of the final product is low due to a prolonged curing period.

Average values from analyses of the raw material and final product found after 15 or more experiments are given in Table 35.

THE FINAL REVISED PLAN OF THE PLANT

In the proposed alteration of the plant the existing equipment will be utilized as much as possible in order to cut expenses to a minimum.

The major points of the revision are as follows:

1. The running hours of the grinder will be 6 hr per day.
2. Ground material is to be sent directly to the digester, without passing through the storage bin.
3. The improved digester consists of 2 cells that are separated by a vertical wall, with each cell having 3 decks as in the original digester. One of the two cells will be fed with raw ground refuse on the first day, and the other one, on the second day. In this way, each cell will be fed with raw ground material every second day. The material in each cell, while constantly stirred, will come down from the 1st deck to the 2nd deck and then reach the 3rd deck. Thence the material will be sent back to the 1st deck by the circulating conveyor. The de-

tention period of material in each cell will be 2 days. The first day will be principally for the feeding and circulation of material while the second day will be for the circulation of material, only.

4. It was originally intended to digest 20 tons of material a day. However, the driving shafts of the cells are the same ones that were installed in the original digester and that as previously explained, have proved to be too weak. Therefore, the quantity will be cut to 15 tons per day.

5. Ribbon-screw type plows will be adopted for the improved digester. The maximum quantity of material to be moved forward is fixed at 15 tons per 6 hr period. Therefore, it is assumed that if the driving motor of the digester is operated continuously for 24 hr, any portion of the material will pass a definite section in the digester four times a day.

Based on the revised plan and computed from the data collected in the experiments, the flow diagrams may be synopsized as follows: For the case of raw nightsoil mixed with refuse it was expected, from experiments, that there would be a 30% reduction of initial organic materials in the digestion stage and 10% reduction in the curing stage. From 15 tons of raw refuse, moisture 40%, from 33,400 persons, 12 tons of refuse at moisture 45% was selected for grinding. The 12 tons of ground refuse at moisture of 45% consisted of 5.41 tons of moisture, 3.08 tons of organic matter and 3.52 tons of inorganic matter. To this was mixed 4.8 tons of nightsoil at moisture 97.2% (moisture 4.666 tons; organic matter 0.072 tons; inorganic matter 0.062 tons) to produce 16.8 tons of mixed material at moisture 60%. The digestion left 10.5 tons of primary compost at moisture 45%. (moisture 4.73 tons; organic matter 2.20 tons; inorganic matter 3.582 tons) Curing produced 9.1 tons of finished compost (moisture 3.648 tons; organic matter 1.890 tons; inorganic matter 3.582 tons) at a moisture content of 40%. The same amounts and constitutent ground refuse was mixed with 4.5 tons of digested sludge of nightsoil (moisture 4.230 tons; organic matter 0.198 tons; inorganic matter 0.072 tons) at moisture 94% from 43,500 persons to produce 16.5 tons of mixed material at moisture 58.5. Digestion produced 10.7 tons of primary compost (moisture 4.810 tons; organic matter 2.295 tons; inorganic matter 3.592 tons) at moisture 45%. After curing there was 9.26 tons of finished compost (moisture 3.705 tons; organic matter 1.965 tons; inorganic matter 3.592 tons) at moisture content of 40%. (In these diagrams the quantity of mixed material is fixed so that its moisture content is 60% or less when raw nightsoil is used and 58% or less when digested sludge of nightsoil is used.)

COMPOSTING OF NIGHTSOIL WITH REFUSE AND FERTILIZING VALUES OF COMPOST

If the moisture of the raw material is over 60%, good compost cannot be produced. It has been proven in the experiments with the pilot plant previously described that by weight, the maximum quantity of nightsoil to be added to refuse in composting is only 2/5 of the amount of compostable refuse, the moisture content of which is 40%.

In Kobe, the amount of nightsoil per capita per day is 1.1 kg, while the amount of compostable refuse per capita per day is 0.36 kg. This means that it is impossible to compost the total quantity of nightsoil produced with the total quantity of refuse produced in Kobe. However, if the nightsoil is anaerobically digested, its overflow liquor can be stabilized by other biological methods

and thus sludge that remains is about 10% by volume of raw nightsoil. Therefore, it is possible to compost the total digested nightsoil sludge produced in Kobe with the total compostable refuse produced. Moreover, digested nightsoil sludge is not only a more valuable fertilizer than raw nightsoil but also is easier to handle.

For reference, a comparative table of the fertilizing ingredients of raw nightsoil, digested nightsoil sludge, and vacuum-filtered digested nightsoil sludge is given in Table 36.

At Kobe City anaerobic nightsoil digestion plant, digested nightsoil sludge is dewatered by a continuous flow type centrifuge, and then piled up for composting.

TABLE 36.—COMPARISON OF DIGESTED NIGHTSOIL SLUDGE WITH RAW NIGHTSOIL [a,b]

Constituent	Raw nightsoil	Digested nightsoil sludge	Vacuum filtered digested nightsoil sludge (Elutriated)	Centrifuged digested nightsoil sludge (Piled up for 10 days).
Solids	2.8	6.0	20.0	40.0
Volatile solids	53.6	73.2	75.0	75.0
Nitrogen	17.1	7.5	2.2	4.0
Phosphorus (as P_2O_5)	4.6	10.0	7.7	10.0
Potash (as K_2O)	4.3	2.0	0.4	0.25
pH	8.7	7.9	7.9	7.9

[a] See values in percentages
[b] Dry basis except solids

Fertilizing Value of Kobe Compost.—The following data were supplied by the Kyoto University Laboratory which conducted pot tests at the request of the writer, on the compost produced at Kobe Composting Plant.

Place:	Kyoto University
Species of plant:	Wheat, "Norin 26."
Soil used:	Kyoto-University Soil
Pots used:	Wagner pots (15 cm dia.)
Period:	Nov. 25, 1956.-May 30, 1957.
Fertilizers used:	Potassium Sulphate as K_2O, 0.5 g per pot
	Calcium-Superphosphate as P_2O_2, 0.5 g per pot
	Ammonium Sulphate as N, 0.5 g per pot
	Compost, 150 g per pot

Table 37 shows clearly that the compost is effective as an organic fertilizer.

SUMMARY AND CONCLUSIONS

Since 1955, several years have been spent in pilot plant research and the construction, operation and alteration of the proto-type plant.

In order to treat such heterogenous material as refuse, it is necessary to make the present digester stronger. In the course of revising the mechanical digester it was found that the easiest way to stir and move material in the digester was to use plows, to the ends of which screw-type ribbons were fixed. Of course a rotary-kiln digester, as that of the Dano-type, will serve this purpose. In order to cut both the operation cost and the capital cost, it is advisable to adopt the following variation of the outdoor windrows process: Pile, stir and mix ground material in a closed structure that prevents the emission of bad odor. Therefore, it is not asserted that a plant of the Kobe type alone is the standard equipment for all compost operations. The operation units of the composting process should differ according to the environment of the plant. The Dano digester treats non-ground material and usually produces good compost. For a mechanical digester such as the Kobe plant, it is absolutely necessary to grind raw material. A new grinder, that has been designed from the practical experience gained in operating the original digester is being used. Nevertheless, studies must continue to find an effective way to cope with the wear of the hammer plates.

TABLE 37.—WEIGHT OF THE PLANT HARVESTED[a]

Fertilizers used	Part	Pot 1	Pot 2	Pot 3	Average
$K_2O+P_2O_5$	Yield	5.0	4.7	4.8	4.8
	Leaves and stem	5.3	4.5	5.0	4.9
$K_2O+P_2O_5$+Compost	Yield	7.2	7.2	6.0	8.8
	Leaves and stem	8.0	8.0	5.5	7.0
$K_2O+P_2O_5+N$	Yield	6.2	5.1	10.2	7.2
	Leaves and stem	5.1	5.0	9.0	6.4
$K_2O+P_2O_5+N$+Compost	Yield	12.0	13.5	13.1	12.9
	Leaves and stem	9.6	9.5	9.3	9.5

[a] All values in grams.

Cities of Japan are densely populated and lack space. Therefore, composting plants to be built in a city should be as compact as possible and issue no obnoxious odor. Accordingly, it is necessary for plants in the city area to adopt the high-rate composting process in which a mechanical digester is used. In Japan, after the construction of the Kobe plant, five more cities built mechanical composting plants. Each treats 30 tons of material per day, and two are using Dano digesters.

The total annual expenditure of the Kobe plant is larger than its annual returns. However, the results of the test prove that the compost is very effective as an organic fertilizer.

In Japan it will be advantageous for a city with a population of approximately 100,000 and surrounded by farming areas to have a composting plant of this kind. Of course compost is effective for vacant land just brought under cultivation. In Japan, however, it is difficult to find this sort of land in the vicinity of a city.

One objective of this research program was to find the maximum quantity of nightsoil that can be disposed of by composting a mixture of refuse, and

nightsoil. As mentioned previously, it has become clear that the composting of this mixture will bring forth no satisfactory overall solution. The nightsoil produced by Japanese households is composed of feces and urine, and its moisture content is about 97%. If only feces are added to refuse, the mixture can be composted. Sludge produced at a sewage disposal plant or a nightsoil disposal plant can be composted with refuse only when the water content of the mixture is 60% or less. Sludge alone can be composted if it is dewatered to the extent that its water content is 60% or less and it is in a granular state.

The process of anaerobic digestion is practiced in Japan for biological disposal of nightsoil. There are anaerobic nightsoil digestion tanks, capable of treating 180 cu m of nightsoil per day, on the premises of Kobe's central sewage disposal plant. The effluent from the tanks is mixed with sewage and the mixture is treated by the activated sludge process. Digested sludge is dewatered by a continuous flow centrifuge until its water content is 75%. The dewatered digested sludge goes through the process of air drying for about a day and is then piled for thermophilic composting. The final product is used as a fertilizer by farmers. Under this treatment, thermal death of helminth ova that are found in sludge is possible even if nightsoil is digested anaerobically at a high rate. Because the B.O.D. and moisture content of nightsoil in Japan is very high (B.O.D. 10,000 ppm), the following may be an effective method for disposing of large quantities of nightsoil: Treat aerobically the mixture of sewage and effluent of the nightsoil digested anaerobically at a high rate and compost the dewatered sludge of the mixture alone.

ACKNOWLEDGMENTS

The project has been financed by Kobe City (Mayor, C. Haraguchi, D. E., Deputy Mayor, T. Miyazaki and Director, S. Inagaki, Construction Bureau) and supported by the National Government. WHO provided the grinder and literature and assigned J. R. Snell to Kobe as the consultant. The designing of the plant and the supervision of the construction work were performed by staff members of the Sewage Department, Construction Bureau, Kobe City Office. The chemical analyses were done by H. Nakagawa, a chemist of the Sewage Department. Valuable suggestions were given by Snell and P. Bierstein, regional advisor, Regional Office, WHO, Manila.

The writer is especially grateful to William G. Turney of the firm of John R. Snell Engineers, Inc., for his efforts in improving the presentation of the material in this paper. Mr. Turney introduced considerable clarity into the manuscript and has made the paper easier for all engineers to study.

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Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

DISCUSSION

Note.—This paper is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SA6, November, 1961.

INVESTIGATIONS ON THE CORROSIVE BEHAVIOR OF WATERS

Closure by Werner Stumm

WERNER STUMM,[4] M. ASCE.—The valuable contribution by James Girand, F. ASCE, is appreciated. Such exchange of information is helpful in orienting others to the type of problems encountered in the quantitative evaluation of corrosion in water works practice. There appears to be general agreement that the chemical composition of water cannot be correlated reliably with its corrosiveness to iron. It has not been possible to corroborate in field investigations many of the well known concepts of corrosion control (for example, the adjustment of pH and Ca C03 saturation).

Girand states the lack of correlation between laboratory work and actual observations of pipes more than 50 yr old. The writer has attempted to show that the mode of corrosive-inhibitive control may significantly change with the time of exposure. It is, of course, almost impossible to predict the corrosive effect of a water over the life span of a pipe from data obtained in tests of but a few days duration. The development of a field method for quantitatively measuring the extent of corrosion over extended periods of time, usuable at any place in the distribution system and under different conditions of flow, would help to solve the current problem. A reasonably good approximation of field conditions can be obtained if representative test specimens are exposed to the flow through a cell (similar to the one used by Girand) which is directly connected to a distribution system.

The flow cell should be designed in such a way that the hydraulic conditions (velocity of flow) can be controlled. The numerous parameters (for example, various flow velocities, various corrective water treatment procedures) which should be investigated for long time periods (preferably years), however, render such field methods extremely costly and time consuming. Unfortunately, only a few water supply plants would have the personnel and the facilities to perform such tests with the necessary thoroughness.

The writer is fully in accord with Girand's suggestion that valuable observations can be made by determining the hydraulic carrying capacity of water mains before and after cleaning (14) (26) (27).

An early survey of the New England Water Works Association (28), based on tests in nineteen cities, reveals that the average actual loss in capacity of tar coated cast iron pipes after 30 yr of service was 52%. Although it can be assumed that the loss in carrying capacity is caused primarily by direct or indirect effects of corrosion and tuberculation, other causes also must be considered. The higher the corrosion rate, the larger are the extent of electrochemical changes and thus the pH increase at the cathodic areas of

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[a] November 1960, by Werner Stumm (Proc. Paper 2657).

[4] Assoc. Prof. of Applied Chemistry, Harvard Univ., Cambridge, Mass.

the corroding metal. Scale deposition of $CaCO_3$ or $Mg(OH)_2$ is, therefore, frequently the indirect result of corrosion. It has been demonstrated that waters that are significantly oversaturated with $CaCO_3$ do not deposit $CaCO_3$ at non-corroding (for example, stainless steel) surfaces (19).

It is common practice to depict pipe line capacity in terms of the Hazen-Wolliams coefficient, C. The coefficient C can be determined from the rate of flow through a pipe under a measured head loss. The coefficient is a measure of hydraulic capacity. Other procedures, such as the determination of the velocity factor (average velocity divided by the velocity at center) may be used for estimating the carrying capacity.

It is believed that the large financial loss resulting from rapid capacity reduction of pipe system due to corrosion warrants the systematic determination of the rate and cause of the loss in water supply distribution systems. The collection of a large number of data on changes in carrying capacity with time in various cities using different water qualities would be a great aid in an attempt to correlate water quality with corrosiveness.

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Errata.—page 29, Table: Analysis, Ca^{+2}, Mg^{+2}, Alkalinity in mg per liter as $CaCO_3$

Corrosion rate in mg per dm^2day
Buffer capacity in milliequivalents per pH
note b, Nitrate

page 32, last line of 2nd paragraph:

Change Fig. 2 to Fig. 3
last line of page:
Change Fig. b to Fig. 3b.

page 35, Eq. (5) should read:

$$\rho = 2.3 \left[\frac{(H^+)\ (Alk)}{(H^+)+2K_2} \left(\frac{(H^+)}{(H^+)+K_1} + \frac{K_2}{(H^+)+K_2} \right) + (H^+)+(OH^-) \right]$$

page 38, Figure: Change caption Fig. 6 to Fig. 7
page 39, Figure: Change caption Fig. 7 to Fig. 6

RADIOLOGICAL CURRICULA FOR SANITARY ENGINEERS[a]

Discussion by E. J. Kilcawley and George B. Morgan

E. J. KILCAWLEY,[5] F. ASCE.—The authors are to be congratulated for the preparation of this clear, concise report on the urgent need for the development and the administration of adequate training programs for the protection of public health through the understanding and control of environmental pollution by radioactivity.

Probably no single event in our development has ever created a more dynamic and far reaching effect than the discovery of nuclear fission and the establishment of the atomic energy industry. Its rapid expansion goes far beyond the most optimistic concepts of its founders and affects, to some extent, every phase of modern living. Because its further development has great potentialities for service in the world, all science and engineering students must be given the opportunity to pursue well-balanced courses of study in nuclear science and its application.

Radiation hazards are potentially involved in every phase of environmental control. The mining and conditioning of reactor fuels, the operation of nuclear reactors for power and other purposes, and the wide spread and increasing use of isotopes introduce environmental problems of a new and different type. The presence of radioactive material and the attendant radioactivity in the environment not only create unique problems, but, at the same time, complicate the conventional problem of environmental control.

Radioactive pollutants occur in particulate, liquid, and gaseous form. The quantity and level of activity varies with the source. Some of the materials formed in nuclear reactors are the most potent poisons known. They are several million to several billion times as toxic as chlorine. These pollutants cannot be detected by any of the human senses and, with the single exception of time, there is no known method of neutralizing the radioactivity. They are serious pollutants of the air, water, food, vegetation, and the ground itself. In the case of fallout, the pollution level varies with the fallout or rainout rate. The heaviest pollution, in the past, has followed the testing of nuclear weapons. These tests release tons of radioactive dusts and gases that are transported in the atmosphere and produce fallouts and rainouts all over the earth.

Exposure to radiation causes both physical and genetic damage to the human body through the process of ionization. The affects of exposure to even low levels of radiation are cumulative and irreversibe on the genes. The

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[a] March 1961, by J. E. Kiker and F. W. Gilcreas (Proc. Paper 2767).

[5] Prof. and Head, Div. of Environmental Engrg., Rensselaer Polytechnic Inst., Troy, N. Y.

damage, therefore, occurs in succeeding generations. These are problems for the medical profession.

The processing of fuels, the operation of nuclear reactors, the safe transportation of isotopes, the disposal of waste products, and environmental safety are problems for teamwork of a group of specially trained professionals. These nuclear operations involve radioactive materials and radiation. The only defense against radiating substances as outside the body sources is to prevent or reduce to a minimum their entrance into the body. Therefore, it follows that the air we breathe, the food we eat, the water we drink must, at all times be free from heavy pollution. Furthermore, the permissible time of body exposure to radiation must be limited in proportion to the intensity and character of the radiating elements involved. Radiation protection, therefore, involves the use of highly efficient methods of control in the source of pollution.

Obviously, environmental safety for the mass populations of our communities requires the professional service of many specialists operating as a team. Each is equally important, but it is essential that the problem be approached as a whole.

Although much effort has been spent to provide the opportunity for the education and training of qualified personnel for service in the various areas of radiation control, there is at present, a serious shortage of qualified people. It is reported that approximately one to two thousand highly trained specialists are currently needed. These must be supported by about twice that number of skilled technicians.

The engineering profession can make a considerable contribution toward a clearer understanding of both the synthesis and analysis of the problem to provide more effective control measures. However, for effective service in this area of environmental control, the engineer must have a more thorough background in science. This science instruction must include not only basic chemistry, biology, and mathematics, but also micro and radio chemistry, micro and radiological biology and ecology.

The writer is in full agreement with the authors that the development of a rational curriculum is not simple. Established curricula in engineering are already overcrowded and course requirements for specific degrees are such that time cannot be made available for the science courses essential to this and other areas of environmental control.

The positive approach to environmental control in our complex changing society requires the services of many professional groups. In fact, the comlexity of problems in each phase of total control requires the professional service of a team of specialists. The representative of engineering on this team occupies an important and responsible position. His contributions and decisions must be based on a knowledge of all of the sciences involved, together with an understanding of the engineering problems that are applicable. This requires a broad educational experience and, at the same time, a greater proficiency in one or more phases of the overall problem.

In recognition of these rapidly changing conditions together with the breadth in understanding required of engineers in the future, Rensselaer Polytechnic Institute (R.P.I.) established in 1958 a new undergraduate curriculum leading to a Master of Science in environmental engineering. At the same time, selected sequential courses leading to the degree, Master of Science in Environmental Engineering were offered. This program was extended in 1959 to pro-

vide instruction leading to the Doctor of Engineering and Doctor of Philosophy.

To provide both breadth and depth, courses offered in various departments of the schools of science, engineering, and humanities and social sciences are chosen to form a suitable program. Study plans are flexible and every effort is made to satisfy the students particular needs. The objective in the arrangement of courses is to achieve, to the greatest degree possible, an integration of disciplines. At the same time, opportunity is offered for a deeper or more concentrated study in the area of air pollution control, water resources development and utilization, pollution control, and radiological health.

The degree, Master of Science, is intended for those students who are preparing for technical and professional responsibilities in engineering organizations. Candidates are expected to have the Baccalaureate in engineering or science. However, elementary courses in physical and organic chemistry and in biology are required. Ordinarily, the requirements for this degree may be completed in one year of residence provided entrance requirements are fully satisfied. The degree, Doctor of Science, is intended for those candidates who wish to follow a career of professional practice. The degree, Doctor of Philosophy, is intended primarily for those who desire to make teaching and research a career. Study leading to either of these degrees may be undertaken by students who have a high level of ability and achievement in graduate work.

GEORGE B. MORGAN.[4]—The sanitary engineer in the field of radiological health needs some background in human anatomy and physiology. Usually one survey course is sufficient and should cover mammalian anatomy, cells, reproduction, and basic physiology. Many universities have such a course, especially those with a school of nursing. As stated in the paper, the sanitary engineer is expected to be the chief staff officer to the physician. To be of greatest value in this position he should be able to communicate with and understand the terminology of the physician. Also the sanitary engineer is expected to use and understand the permissible levels of radioactivity in the environment. These concentrations are based on the absorption and translocation of radionuclides in the body.

The writer would like to see some provision made for non-engineering students to enter the field of radiological health. A person with a strong background in science (chemistry, biology, and physics), who received further training in sanitary science and radiological health, will prove invaluable on the staff of any rad health program. Such a person does not need background courses such as those required for all engineers but does need the basic sanitary engineering courses. For these exceptions the degree of Master of Sanitary Science may be advisable.

Courses in basic electronics should be available for the doctoral candidate. A knowledge of the system by which radiation is determined is of extreme importance for interpretation of data. Also for planning certain investigative work in Rad Health, this knowledge is necessary.

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SANITARY ENGINEERING COMES OF AGE[a]

Discussion by Harold Farnsworth Gray and Kenneth D. Kerri

HAROLD FARNSWORTH GRAY,[3] F. ASCE.—It is incomprehensible to the writer that McGauhey should make the statement "from its beginnings in the 1920's sanitary engineering as a specialty soon developed into an undergraduate option in the civil engineering curriculum."

At the University of California the sanitary engineering option began in 1905, when Charles Gilman Hyde, H. M. ASCE, came to Berkeley and began his brilliant and inspired teaching career; the writer was one of his first students, so he remembers. Hyde was a disciple of Sedgwick at Massachusetts Institute of Technology, Cambridge, Mass. Both Sedgwick and Hyde were "biologically oriented," and impressed this bias on their students. They urged the experimental approach to the solution of problems, and also stressed the desirability of a broad cultural background to be acquired in the humanities by self-study, since there was no time for such studies in the engineering curriculum.

In 1907, graduates called themselves "sanitary engineers," and C. G. Gillespie, F. ASCE, and the writer went directly into experimental research on water filtration under W. W. DeBerard, F. ASCE, and Langdon Pearse.

The writer does not remember an over-emphasis on structural design; students had time to take a course in bacteriology, and studied Sedgwick's "The Principles of Sanitary Science and the Public Health" (1901) which was definitely biologically oriented.

The state of the art at that time did not require several years of graduate study; but in 1910 the writer found it desirable to put in nearly three years of graduate study, in sanitary engineering, parasitology, and public health. Today, at least two years of graduate study appears to be the minimum possible in which to encompass the core of essential studies.

When one realizes that an attorney normally requires seven years of study, and a physician from ten to fourteen or more years, it does not seem to be unreasonable, if an engineer is to consider himself (and to be considered) a professional practicioner of the art, that he spend six years of study.

Without taking any credit from subsequent teachers such as Langelier, Fair, and others, it is well to remember the earlier teachers such as Sedgwick, Hyde, Whipple, and others. They pointed out the way; they laid the foundations.

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[a] May 1961, by P. H. McGauhey (Proc. Paper 2806).
[3] San. and Hydr. Engr. (retired), Oroville, Calif.
[4] Asst. Prof. of Civ. Engrg., Sacramento State Coll., Sacramento, Calif.

KENNETH D. KERRI,[4] A.M. ASCE.—P. H. McGauhey has emphasized the lack of student enrollment in graduate sanitary engineering programs. One of the best methods of directing and encouraging students to participate in a graduate program is a properly taught undergraduate sanitary engineering course.

The writer believes the course should be interesting and informative in order that the undergraduate civil engineering student will realize the capabilities of a sanitary engineer and the scope of the field. Many students suffer the misconception that sanitary engineering consists solely of sloshing around in sewers. The undergraduate course should not attempt to produce qualified sanitary engineers. Many prospective sanitary engineers are discouraged as undergraduate civil engineers because the professor "snowed" the student with material beyond the scope of his comprehension and overburdened him with excessive "busy" work. The course should prepare the student for graduate school and also to enter the field as a professional engineer. Subject matter in areas where civil engineers in the community practice sanitary engineering (subsurface disposal systems) should be emphasized and students may be effectively educated concerning community problems (fluoridation and bond issues).

Some colleges and universities are located a considerable distance from metropolitan areas where the majority of sanitary engineering consultants are located. The undergraduate sanitary engineering program provides the community with a sanitary engineer familiar with local sanitary and public health problems. Also available are the research facilities of the sanitary engineering laboratory to facilitate economical solutions to local problems.

In conclusion, the writer feels it is the obligation of the undergraduate curriculum to encourage and direct qualified students toward a graduate school capable of producing competent sanitary engineers.

SANITARY ENGINEERING EDUCATION IN A CHANGING WORLD[a]

Discussion by K. J. Ives

K. J. IVES,[25] M. ASCE.—In Great Britan the graduate civil engineer rarely engages in formal study after graduation but is usually anxious to attain corporate membership of the Institution of Civil Engineers (A.M.I.C.E.). This requires a minimum of 3 yr practical training, including site and design office work of at least 1 yr each. Consequently, the graduate professional civil engineer is qualified by 3 yr university training and 3 yr practical training. This applies generally to sanitary engineers, although other specialist institutions are available (Institution of Public health Engineers, Institution of Municipal Engineers, Institution of Water Engineers, Institution of Sewage Purification, Royal Society of Health) that do not carry the status of the Institution of Civil Engineers.

Graduate courses are available in sanitary engineering (Public Health Engineering) at some universities in Great Britain, but they are not well attended by engineers from the United Kingdom. The content of the courses is comparable with Masters' programs available in the United States but are not strongly supported by United Kingdom engineers for a number of reasons:

1. The desire of young men on graduation to commence training for A.M.I.C.E.: the sine qua non of the professional civil engineer.
2. The reluctance of local or central government organizations to release and finance mature men to attend such courses.
3. The fact that the courses award a diploma on completion: this does not carry the status of a Master's degree in many people's opinion.
4. The lack of recognition by employers (usually local government) of qualifications beyond the Bachelor's degree and corporate membership of a professional institution. At University College, London, this situation has been recognized and met by incorporating the fundamentals of sanitary engineering into the final year of the honors civil engineering degree course, B.Sc.(Eng.), so that the subsequent practical training can be supported by the foundations of theory. This incorporation of "sanitary science" into undergraduate training dates from 1897, being directly influenced by Edwin Chadwick and initially financed by funds from the Chadwick Trust derived from his will.

Research in sanitary engineering in a British university usually leads to a Master's or Ph.D. degree. The number of United Kingdom engineers qualifying in this manner is disappointingly small. The reasons listed before (except number 3) apply equally here; furthermore, there is little outlet for the

[a] July 1961, by Daniel A. Okun (Proc. Paper 2862).

[25] Lecturer, Dept. of Civ. Engrg., Univ. College, London, England.

research trained and qualified sanitary engineer in the United Kingdom. The universities have little capacity in this respect, and the two principal research organizations in this field: the government sponsored Water Pollution Research Laboratory and the water industry's Water Research Association employ no civil or sanitary engineers whatever. Thus, of the six British engineers who gained the Ph.D. in Public Health Engineering from University College, London, in the last decade, two are in University teaching, one is teaching structural engineering in a college of advanced technology, two are in management in industry (paper and petrochemicals) and one entered the ministry of the Church!

A current trend to be deplored is the neglect of sanitary engineering research because of the development of sanitary chemistry and biology which lend themselves more readily to laboratory research. Research by qualified chemists or biologists in sanitary science is an admirable adjunct to, not substitute for, sanitary engineering research. This is the more to be deplored when first class engineers are led to carry out research in chemistry or biology that could be more competently done by graduates in those fields of study.

With regard to the philosophy of sanitary engineering education, it is surely better to develop the intellect of the student by study of the subject in depth. This training of the mind should serve the graduate to explore any subject either of science or humanities as required by his interests or profession after he has left University. In this respect the writer would agree wholeheartedly with Okun that injections of odd courses in the humanities are of little value in the student's curriculum.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU); and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2703 is identified as 2703(ST1) which indicates that the paper is contained in the first issue of the Journal of the Structural Division during 1961.

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NOVEMBER: 2637(ST11), 2638(ST11), 2639(CO3), 2640(ST11), 2641(SA6), 2642(WW4), 2643(ST11), 2644(HY9), 2645(ST11), 2646(HY9), 2647(WW4), 2648(WW4), 2649(WW4), 2650(ST11), 2651(CO3), 2652(HY9), 2653 (HY9), 2654(ST11), 2655(HY9), 2656(HY9), 2657(SA6), 2658(WW4), 2659(WW4)[c], 2660(SA6), 2661(CO3), 2662(CO3), 2663(SA6), 2664(CO3)[c], 2665(HY9)[c], 2666(SA6)[c], 2667(ST11)[c].

DECEMBER: 2668(ST12), 2669(IR4), 2670(SM6), 2671(IR4), 2672(IR4), 2673(IR4), 2674(ST12), 2675(EM6), 2676(IR4), 2677(HW4), 2678(ST12), 2679(EM6), 2680(ST12), 2681(SM6), 2682(IR4), 2683(SM6), 2684(SM6), 2685(IR4), 2686(EM6), 2687(EM6), 2688(EM6), 2689(EM6), 2690(EM6), 2691(EM6)[c], 2692(ST12), 2693 (ST12), 2694(HW4)[c], 2695(IR4)[c], 2696(SM6)[c], 2697(ST12)[c].

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JANUARY: 2698(PP1), 2699(PP1), 2700(HY1), 2701(SA1), 2702(SU1), 2703(ST1), 2704(ST1), 2705(SU1), 2706 (HY1), 2707(HY1), 2708(HY1), 2709(PO1), 2710(HY1), 2711(HY1), 2712(ST1), 2713(HY1), 2714(PO1), 2715 (ST1), 2716(HY1), 2717(SA1), 2718(SA1), 2719(SU1)[c], 2720(SA1)[c], 2721(ST1), 2722(PP1)[c], 2723(PO1)[c], 2724(HY1)[c], 2725(ST1)[c].

FEBRUARY: 2726(WW1), 2727(EM1), 2728(EM1), 2729(WW1), 2730(WW1), 2731(EM1), 2732(SM1), 2733(WW1), 2734(SM1), 2735(EM1), 2736(EM1), 2737(PL1), 2738(PL1), 2739(PL1), 2740(PL1), 2741(EM1), 2742(ST2), 2743(EM1), 2744(WW1), 2745(WW1), 2746(SM1), 2747(WW1), 2748(EM1), 2749(WW1), 2750(WW1)[c], 2751 (EM1)[c], 2752(SM1)[c], 2753(PL1)[c], 2754(ST2)[c], 2755(PL1).

MARCH: 2756(HY2), 2757(IR1), 2758(AT1), 2759(CO1), 2760(HY2), 2761(IR1), 2762(IR1), 2763(HY2), 2764 (ST3), 2765(HY2), 2766(HW1), 2767(SA2), 2768(CO1), 2769(IR1), 2770(HY2), 2771(SA2), 2772(HY2), 2773 (CO1), 2774(AT1), 2775(IR1), 2776(HY2), 2777(HY2), 2778(SA2), 2779(ST3), 2780(HY2), 2781(HY2)[c], 2782 (HW1)[c], 2783(SA2)[c], 2784(CO1), 2785(CO1)[c], 2786(IR1)[c], 2787(ST3)[c], 2788(AT1)[c], 2789(HW1).

APRIL: 2790(EM2), 2791(SM2), 2792(SM2), 2793(SM2), 2794(SM2), 2795(SM2), 2796(SM2), 2797(SM2), 2798 (EM2), 2799(EM2), 2800(EM2), 2801(EM2), 2802(ST4), 2803(EM2)[c], 2804(SM2)[c], 2805(ST4)[c].

MAY: 2806(SA3), 2807(WW2), 2808(HY3), 2809(WW2), 2810(HY3), 2811(WW2), 2812(HY3), 2813(WW2), 2814 (HY3), 2815(WW2), 2816(HY3), 2817(HY3), 2818(SA3), 2819(WW2), 2820(SA3), 2821(WW2), 2822(WW2)[c], 2823(HY3), 2824(SA3), 2825(HY3), 2826(SA3)[c], 2827(HY3)[c].

JUNE: 2828(SM3), 2829(EM3), 2830(EM3), 2831(IR2), 2832(SM3), 2833(HW2), 2834(IR2), 2835(EM3), 2836 (IR2), 2837(IR2), 2838(SM3), 2839(SM3)[c], 2840(IR2)[c], 2841(HW2)[c], 2842(EM3)[c], 2843(ST5), 2844(ST5), 2845(ST5), 2846(ST5)[c].

JULY: 2847(PO2), 2848(SU2), 2849(HY4), 2850(PO2), 2851(HY4), 2852(PO2), 2853(SU2), 2854(HY4), 2855 (PO2), 2856(PO2), 2857(PO2), 2858(SA4), 2859(SU2), 2860(SA4), 2861(PO2), 2862(SA4), 2863(HY4), 2864 (HY4), 2865(HY4), 2866(HY4), 2867(HY4), 2868(PO2)[c], 2869(SA4)[c], 2870(SU2)[c], 2871(HY4), 2872(HY4)[c], 2873(SU2), 2874(SA4).

AUGUST: 2875(WW3), 2876(WW3), 2877(WW3), 2878(SM4), 2879(ST6), 2880(EM4), 2881(SM4), 2882(EM4), 2883(WW3), 2884(EM4), 2885(SM4), 2886(WW3), 2887(EM4), 2888(WW3), 2889(AT2), 2890(AT2), 2891 (AT2), 2892(AT2), 2893(AT2), 2894(AT2), 2895(AT2), 2896(AT2), 2897(AT2), 2898(AT2), 2899(AT2), 2900 (AT2), 2901(AT2), 2902(SM4), 2903(ST6), 2904(ST6), 2905(SM4), 2906(ST6), 2907(EM4), 2908(ST6), 2909 (EM4), 2910(ST6), 2911(EM4), 2912(SM4), 2913(ST6), 2914(WW3)[c], 2915(ST6)[c], 2916(EM4)[c], 2917(SM4)[c].

SEPTEMBER: 2918(SA5)[c], 2919(HW3)[c], 2920(HY5)[c], 2921(SA5), 2922(PL2), 2923(IR3), 2924(HY5), 2925 (HY5), 2926(CP1), 2927(HY5), 2928(IR3), 2929(IR3), 2930(HY5), 2931(CP1), 2932(PL2), 2933(HY5), 2934 (HY5), 2935(HY5), 2936(HY5), 2937(HW3), 2938(CP1), 2939(PL2), 2940(SA5), 2941(SA5), 2942(SA5), 2943 (HY5), 2944(PL2)[c], 2945(CP1)[c], 2946(IR3), 2947(HW3), 2948(IR3), 2949(IR3)[c].

OCTOBER: 2950(PP2), 2951(PP2), 2952(PP2), 2953(ST7), 2954(SM5), 2955(ST7), 2956(ST7), 2957(ST7), 2958 (ST7), 2959(SM5), 2960(EM5), 2961(EM5), 2962(ST7), 2963(ST7), 2964(EM5), 2965(SM5), 2966(SM5), 2967 (ST7), 2968(ST7), 2969(ST7), 2970(ST7), 2971(SM5), 2972(SM5)[c], 2973(EM5)[c], 2974(ST7)[c], 2975(PP2).

NOVEMBER: 2976(HY6), 2977(HY6), 2978(HY6), 2979(PO3), 2980(WW4), 2981(HY6), 2982(HY6), 2983 (HY6), 2984(HY6), 2985(SA6), 2986(SA6), 2987(WW4), 2988(WW4), 2989(WW4), 2990(HY6), 2991 (WW4), 2992(HY6), 2993(PO3), 2994(WW4), 2995(PO3), 2996(HY6), 2997(SA6), 2998(PO3), 2999 (PO3)[c], 3000(HY6)[c], 3001(SA6), 3002(SA6), 3003(SA6)[c], 3004(WW4)[c].

c. Discussion of several papers, grouped by divisions.

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